NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP Report 405

Aggregate Tests Related to Asphalt Concrete Performance in Pavements

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Report 405

Aggregate Tests Related to Asphalt Concrete Performance in Pavements

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

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The program is developed on the basis of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the National Research Council and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are the responsibilities of the National Research Council and the Transportation Research Board.

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FOREWORD

By Staff
Transportation Research Board

This report contains the findings of a study that was performed to identify performance-based procedures for testing and selecting aggregates for use in hot-mix asphalt (HMA) concrete mixtures. The report provides a comprehensive description of the research, including a recommended set of aggregate tests to help materials engineers evaluate and select the aggregates that should contribute to good performing asphalt concrete pavements. Also, the report describes the test methods for the recommended aggregate tests. The contents of this report will be of immediate interest to materials engineers, researchers, and others concerned with the construction and performance of asphalt concrete pavements.

The properties of coarse and fine aggregates used in HMA concrete mixtures are very important to the performance of the pavement system in which these mixtures are used. Because many of the currently used tests were developed to characterize aggregates empirically without, necessarily, any relationship to the performance of the HMA in the pavement system, their use has contributed to improper aggregate selection that has led to less than desired pavement performance.

Under NCHRP Project 4-19, "Aggregate Tests Related to Asphalt Concrete Performance in Pavements," Auburn University was assigned the task of recommending a set of aggregate tests that relate to the performance of HMA concrete used in pavement construction. To accomplish this objective, the researchers reviewed relevant domestic and foreign literature; identified aggregate properties that influence the performance of HMA concrete in pavements; identified and evaluated, in a laboratory investigation, the aggregate tests currently used in the United States and other countries as well as potential new aggregate tests to measure HMA performance-related properties; and recommended a set of performance-based aggregate tests. The report documents the work performed under Project 4-19 and discusses the linkage between aggregate tests and the performance of asphalt concrete pavements.

The recommended set of aggregate tests can be used to evaluate and select aggregates for use in HMA concrete mixtures. The report includes descriptions of those recommended test methods that are not currently being used in the United States. These test methods will be particularly useful to highway agencies and are recommended for consideration and adoption by AASHTO as standard test methods.

CONTENTS

AUTHOR ACKNOWLEDGMENTS

The research reported herein was performed under NCHRP Project 4-19 by the National Center for Asphalt Technology (NCAT) at Auburn University, Alabama. The work was carried out under the direction of Prithvi S. Kandhal, Associate Director of NCAT, who served as principal investigator, and Frazier Parker, Jr., Professor of Civil Engineering, who served as co-principal investigator. Graduate students Yiping Wu and Cynthia Lynn assisted the principals in carrying out the research. Richard D. Barksdale, Professor Emeritus of Civil Engineering at Georgia Tech, and Peter Malphurs served as consultants.

AGGREGATE TESTS RELATED TO ASPHALT CONCRETE PERFORMANCE IN PAVEMENTS

SUMMARY

Because aggregates constitute about 94 percent by weight of hot-mix asphalt (HMA), the properties of coarse and fine aggregates used in HMA are very important to the performance of the pavement system in which these mixtures are used. Many of the currently used tests were developed to characterize aggregates empirically without, necessarily, any relationship to the performance of HMA in pavement systems. Therefore, research is needed to evaluate how well existing aggregate tests and potential aggregate tests assess pavement performance. The objective of this research project is to recommend a set of aggregate tests that can be used to evaluate the performance of HMA used in pavement construction.

The following steps were taken to complete this research project:

1. Identification of the performance parameters of HMA that may be affected by the properties of the aggregates;
2. Identification of the aggregate properties that influence the performance parameters established in Step 1;
3. Identification and evaluation of the aggregate tests currently used in the United States and other countries, which measure the properties identified in Step 2;
4. Identification of potential tests for measuring those aggregate properties for which no suitable test method was identified in Step 3;
5. Conduct of a laboratory investigation to evaluate aggregate tests (identified in Steps 3 and 4) in terms of how they can be used to evaluate the performance of HMA; and
6. Recommendation of a set of aggregate tests that can be used to evaluate the performance of HMA in pavement on the basis of the results of the laboratory investigation in Step 5.

The following three HMA performance parameters were considered in this study: (a) permanent deformation; (b) fatigue cracking; and (c) raveling, popouts, or potholing. Permanent deformation resulting from moisture-induced damage or stripping of HMA was also considered. Frictional resistance of HMA was not considered because of limited project funding. The following six aggregate properties, which are related to HMA performance parameters, were included in the laboratory research plan:

- Coarse aggregate particle shape, angularity, and surface texture;
- Fine aggregate particle shape, angularity, and surface texture;
- Plastic fines in the fine aggregate;
- Toughness and abrasion resistance;
- Durability and soundness; and
- Characteristics of P200 (material passing No. 200 sieve).

Aggregate gradation and size, although related to HMA performance, were not included in the research plan because standard sieve analysis methods already exist.

The research team selected a variety of aggregates in order to obtain a wide range of test values for the specific property to be evaluated. Field performance history was factored into the selection process for the toughness and abrasion study and the durability and soundness study.

After measuring the specific property (e.g., particle shape, angularity, and surface texture) of each aggregate by using different aggregate tests (both tests commonly used in the United States and tests new to the United States), these aggregates were incorporated in HMA mixtures. Mix validation tests were then conducted to measure pertinent mix performance properties (e.g., permanent deformation and fatigue cracking). Mix validation tests included the Superpave shear tester (of permanent deformation and fatigue cracking), Georgia loaded wheel tester (of permanent deformation), Hamburg wheel tracking device test (of stripping), and AASHTO T 283 (procedures to test stripping).

The following aggregate tests, which can be used to evaluate HMA performance parameters (identified in parentheses), are recommended for use in evaluating aggregates for HMA pavements:

1. Gradation and size (permanent deformation and fatigue cracking);
2. Uncompacted void content of coarse aggregate (permanent deformation and fatigue cracking);
3. Flat or elongated particles (2:1 ratio) in coarse aggregate (permanent deformation and fatigue cracking);
4. Uncompacted void content of fine aggregate (permanent deformation);
5. Methylene blue test of fine aggregate (permanent deformation resulting from stripping of HMA);
6. Particle size analysis of P200 material for determining D60 and D10 sizes (permanent deformation resulting from traffic loads as well as stripping) (Note: D60 and D10 are particle sizes in mm that have 60 percent and 10 percent passing, respectively);
7. Methylene blue test of P200 material (permanent deformation resulting from stripping of HMA);
8. Micro-Deval test (raveling, popouts, or potholing); and
9. Magnesium sulfate soundness test (raveling, popouts, or potholing);

Test protocols have been developed in AASHTO format for all new aggregate test methods and are included as Appendix D.

The findings of this project are based primarily on laboratory experiments—the recommended aggregate tests have not yet been validated in the field. Therefore, a field validation plan is provided in this report. The recommended field validation plan will also be helpful in establishing minimum and/or maximum test values for specification purposes. The field validation plan should be executed as soon as possible.

CHAPTER 1

INTRODUCTION AND RESEARCH APPROACH

PROBLEM STATEMENT AND RESEARCH OBJECTIVE

The properties of coarse and fine aggregates used in hot-mix asphalt (HMA) affect the performance of the pavement system. Often pavement distress, such as stripping and rutting, can be traced directly to improper aggregate selection and use. Proper aggregate selection is necessary to attain desired performance.

Many current aggregate tests were developed to characterize an aggregate empirically without, necessarily, the tests having strong relationships with the pavement performance of the final product. Widespread use, familiarity, and a historical database have increased the popularity of some tests; however, the highway industry would be better served by other tests that would provide a clearer relationship to performance. Thus, research is needed to evaluate existing aggregate tests, identify new tests that relate to performance, and develop better procedures for testing and selection of aggregate used in various HMA applications. This research is concerned with HMA used in the different layers of flexible and rigid pavement systems, including overlays.

The objective of the research is to recommend aggregate tests that relate to the performance of HMA used in pavement construction. This research included the evaluation of existing aggregate tests to assess their ability to predict pavement performance and, where this predictive ability or clear relationship to performance was lacking, the development of new tests.

SCOPE OF STUDY

This study consisted of two phases. Phase I comprised six tasks, and Phase II consisted of three tasks.

Phase I

The tasks in Phase I were as follows:

- Task 1—Identify those performance parameters of HMA used in pavement construction that may be affected by the properties of the aggregates.
- Task 2—Identify those aggregate properties that influence the performance parameters identified in Task 1 and can be used to predict the pavement performance (including physical, chemical, mechanical, and petrographic characteristics).
- Task 3—Identify and evaluate those test procedures currently used in the United States and other countries for measuring properties of aggregates used in HMA. Determine which tests can be used to measure the properties identified in Task 2.
- Task 4—Identify and evaluate potential techniques for measuring those performance-related properties for which no suitable test method was identified in Task 3.
- Task 5—Develop a research plan (encompassing a laboratory investigation) to evaluate and validate the techniques and test methods (identified in previous tasks) for measuring aggregate properties that relate to pavement performance.
- Task 6—Prepare an interim report that documents the research performed in Tasks 1 through 5 and provides a work plan for the Phase II portion of the project.

Phase II

The tasks in Phase II were as follows:

- Task 7—Execute the research plan, including the laboratory test program, approved in Phase I. On the basis of the results of this work, recommend a set of tests for evaluating aggregates used in different HMA concrete applications.
- Task 8—Develop protocols for the tests recommended in Task 7, in a format suitable for consideration and adoption by AASHTO.
- Task 9—Submit a final report that documents the entire research effort. The report will include a plan for validating the relationship of the proposed tests to pavement performance in the long term and an implementation plan for putting the results of this research into practice.

RESEARCH APPROACH

The research approach for this project included a review of literature (Appendix A); selection of aggregate properties for study; identification, description, and evaluation of current aggregate test procedures; identification, description, and evaluation of potential techniques for measuring aggregate properties; and development and execution of a research plan.

Literature Review

An extensive review of literature was conducted to identify those performance parameters (e.g., permanent deformation and fatigue cracking) of HMA affected by the properties of aggregates. The literature review was also used to evaluate how various aggregate properties affect the performance parameters of HMA (e.g., permanent deformation and fatigue cracking).

The following aggregate properties were evaluated: gradation and size; particle shape, angularity, and surface texture; porosity or absorption; cleanliness and deleterious materials; toughness and abrasion resistance; durability and soundness; expansive characteristics; polish and frictional characteristics; mineralogy and petrography; chemical properties; and characteristics of the material passing 75 μm (No. 200) sieve or the P200 material.

Selection of Aggregate Properties for Study

Those aggregate properties assessed as having a major influence on HMA performance parameters were selected for further study.

Identification, Description, and Evaluation of Current Aggregate Test Procedures

Specifications for aggregates used in HMA construction in the United States and some other countries were reviewed to identify, describe, and evaluate the aggregate test procedures currently used to determine the selected aggregate properties.

Identification, Description, and Evaluation of Potential Techniques for Measuring Aggregate Properties

Potential techniques for measuring the selected aggregate properties were identified, described, and evaluated. Development of new test procedures was considered necessary in case the commonly used tests lacked the predictive ability or clear relationship to HMA performance.

Development and Execution of Research Plan

A research work plan was developed on the basis of the information gathered in the preceding activities. General guidelines were formulated for selection of materials, selection of mix validation tests, design of experiments, and statistical analysis of test data. Detailed work plans were then developed for each of the selected aggregate properties. After the research plan was executed, the test data were analyzed statistically to determine which aggregate tests are related to performance of HMA in pavements. A list of these aggregate tests was then prepared. Protocols were developed in AASHTO format for the recommended new test methods. Because this project was primarily a laboratory effort, a plan was developed for validating the relationship of the proposed tests to pavement performance in the field.

CHAPTER 2

THE AGGREGATES' ROLE IN HMA PERFORMANCE

PERFORMANCE PARAMETERS FOR HMA

The research team's extensive review of literature (Appendix A) revealed that several performance parameters of HMA used in pavement construction are affected significantly by the properties of the aggregate. A brief discussion of each of these performance parameters follows.

Permanent Deformation

Premature rutting (or permanent deformation) of heavy-duty HMA pavements has increased in recent years. Studies conducted by the National Center for Asphalt Technology (NCAT) (*1, 2*) have indicated that the rutting has generally occurred in the top 75 to 100 mm (3 to 4 in.) of HMA pavements. Increased rutting results primarily from high-pressure truck tires and increased wheel loads. Although proper selection of asphalt binder grade minimizes rutting, the properties of the aggregates (e.g., size, gradation, particle shape, angularity, and surface texture) also affect rutting.

Rutting can also occur indirectly from premature stripping of the HMA layer(s) and subsequent deterioration and/or consolidation. Stripping or moisture susceptibility of HMA pavements was identified as a significant problem in the 1970s in the southeastern United States. Since then, more and more states have experienced this problem (*3*). Research conducted under the Strategic Highway Research Program (SHRP) A-003B Project has shown that the physico-chemical surface properties of mineral aggregates are more important than the asphalt binder properties in terms of adhesion and moisture-induced stripping (*4*). Some mineral aggregates are inherently very susceptible to moisture-induced stripping. This research concluded that dust coatings naturally occurring on aggregate surfaces can change the chemistry of adhesion and result in weak bonding between the dust and aggregate surface that leads to attrition of the bonding forces that help maintain the integrity of the HMA mixture.

Raveling, Popouts, or Potholing

Aggregate for HMA must be resistant to degradation during production (i.e., stockpiling, handling, and mixing) and construction (i.e., transport, laydown, and compaction). Tough, abrasion-resistant aggregate will minimize construction control problems and ensure that mix in the pavement has properties as close as possible to design. HMA mixtures must also be resistant to degradation when subjected to traffic. Because degradation of HMA mixtures is primarily controlled by the properties of aggregate, the aggregates must be resistant to the abrading and grinding action of heavy traffic.

The HMA mixture should also be resistant to weathering during its service life. Although coated by an asphalt binder film, aggregates are exposed to weathering action (e.g., wetting and drying and freezing and thawing) when they are cracked or crushed during compaction or when the asphalt film is worn off by traffic. Partial stripping of the HMA layers and/or degradation of the aggregate can also expose aggregate surfaces to the elements. Therefore, aggregates must be sound and resistant to weathering. Aggregates that become weak or disintegrate when exposed to moisture, wetting and drying, freezing and thawing, or heating and cooling are not desirable.

Fatigue Cracking

Fatigue of HMA pavements results from repeated traffic loading and becomes evident in the form of cracking. Stiffer HMA mixtures have greater fatigue life in thick HMA pavements (controlled-stress mode of loading) whereas more flexible HMA mixtures have greater fatigue life in thin HMA pavements (controlled-strain mode of loading). Air void content affects the fatigue life and should be as small as possible (but not less than 3 percent) to obtain optimum fatigue life. The asphalt content of the HMA mixture affects fatigue life. The asphalt content should be as high as possible given the stability (resistance to rutting) and frictional resistance (if applicable) of the mixture. The gradation of mineral aggregate has a significant effect on the stiffness (and, therefore, fatigue life) of the HMA mixture. Dense-graded aggregates are recommended for use in thick HMA pavements to obtain increased mix stiffness; more open-graded aggregates are desirable for thin HMA pavements to obtain lower mix stiffness.

Aggregate particle shape, angularity, and surface texture are also believed to affect the fatigue life of HMA mixtures.

Angular particle shape and rough surface texture are thought to enhance fatigue life. Some types of materials passing the 75 μm (No. 200) sieve (P200 material) may stiffen the HMA mixes excessively, thereby affecting their fatigue life.

Frictional Resistance

Frictional resistance of the HMA wearing course is an important performance parameter for the safety of the motoring public. If frictional resistance deteriorates below a minimum acceptable (safe) level, the pavement cannot serve its desired function. If the mix has been designed properly (i.e., there is no excessive asphalt content), the frictional resistance of the HMA pavement surface depends on the pavement surface texture, which can be categorized as either macrotexture or microtexture. Macrotexture can be obtained by controlling the gradation of the aggregates in the mix. Microtexture depends on the exposed aggregate surfaces and, thus, is dependent on the properties of the aggregate, including mineralogy.

Low-Temperature or Thermal Cracking

Aggregates have minimal effect on low-temperature or thermal cracking, which is controlled primarily by the low temperature properties of the asphalt binder. This is despite the fact that low-temperature cracks are frequently observed through some aggregate particles. However, the aggregate gradation does have an influence on this distress mechanism. If a highly temperature-susceptible asphalt binder is used in a dense-graded HMA mix and in an open-graded HMA mix, the former will have higher mix stiffness and thus will be more likely to develop thermal cracking than the latter.

Because of the considerable influence of aggregate properties on permanent deformation; raveling, popouts, or potholing; and fatigue cracking; these performance parameters were included in the scope of this study. Performance parameters that are not largely influenced by aggregate properties (e.g., frictional resistance and low-temperature cracking) were not included.

AGGREGATE PROPERTIES RELATED TO HMA PERFORMANCE PARAMETERS

A detailed review of literature was conducted (see Appendix A) to identify aggregate properties that are related to HMA performance parameters discussed previously. A brief discussion of these aggregate properties follows.

Gradation and Size

Gradation of the combined aggregate in an HMA mixture significantly affects the performance of the HMA pavement. In dense-graded HMA mixtures, a reasonably dense gradation with adequate voids in the mineral aggregate (VMA) provides improved resistance to degradation during construction and under traffic and improved resistance to fatigue cracking when used in thick pavements. In open-graded mixtures (e.g., open-graded friction course [OGFC] and stone-matrix asphalt [SMA]), it is desirable to have a gradation which gives stone-on-stone contact of the coarse aggregate. Maximum aggregate size is also important. HMA mixtures with large maximum size generally resist permanent deformation better than those with small maximum size.

Although aggregate gradation and size influence HMA performance parameters such as permanent deformation and fatigue cracking, evaluation of these properties was not included in the research plan because an acceptable, standard sieve analysis method already exists to determine gradation and size.

Particle Shape, Angularity, and Surface Texture

Studies (discussed in Appendix A) on the effect of particle shape, angularity, and surface texture of aggregates on the performance parameters of HMA mixtures revealed the following:

- Particle shape, angularity, and surface texture of the coarse aggregate are more critical in open-graded HMA mixtures than they are in dense-graded mixtures.
- Particle shape, angularity, and surface texture of the fine aggregate have more influence on the physical properties of the dense-graded HMA mixtures than on those of the coarse aggregate.
- Angular and rough-textured particles are desirable to obtain HMA mixtures that resist permanent deformation and fatigue cracking. Frictional resistance of the wearing courses is also improved when such aggregates are used in the mixture.
- A high percentage of crushed particles is desirable in gravel aggregates to obtain crushed faces with sharp edges and rough surface texture.
- The presence of flat and elongated aggregate particles is undesirable in HMA mixtures—such particles tend to break down (especially in open-graded mixtures) during production and construction, thus affecting the durability of HMA mixtures.
- According to one study, thin (slabby) aggregates tend to reduce the fatigue life of HMA mixtures.

Because aggregate particle shape, angularity, and surface texture could be used in predicting HMA pavement performance in terms of resistance to permanent deformation and resistance to fatigue cracking, this aggregate property was included in the research plan.

Porosity or Absorption

Mineral aggregates used in HMA mixtures have some porosity and, as such, tend to absorb some asphalt binder. Existing mix design procedures consider the amount of absorbed (ineffective) asphalt binder and compensate by incorporating additional binder in the mix. HMA mixtures containing highly absorptive blast furnace slag aggregate perform satisfactorily when properly designed. If the absorbed binder is not compensated for, the following problems may occur: (a) insufficient binder after absorption leading to raveling, cracking, or stripping; (b) possible premature age hardening of asphalt binder; and (c) construction problems, such as segregation.

Because aggregate porosity or absorption cannot be used for predicting HMA pavement performance, this property was not included in the research plan.

Cleanliness and Deleterious Materials

"Cleanliness" is concerned with coatings on aggregate particles or excess material passing 0.075 mm or No. 200 sieve. "Deleterious materials" include individual particles made up of weak, reactive, or unsound particles.

The adhesion between the asphalt binder and aggregate can be inhibited by the presence of dust and clay coatings on the coarse and/or fine aggregate, which results in stripping of the HMA mixture. In the presence of water, some very fine clayey material may cause stripping by weakening the asphalt binder.

Deleterious materials that inhibit asphalt binder from coating aggregate particles, react with the environment, or do not consist of mineral matter are also detrimental to HMA mixtures. Some examples are clay lumps, friable particles, shale, coal, glassy particles, free mica, and vegetation. The presence of free mica in aggregate has presented performance problems in some states. It is difficult to quantify the effect of many miscellaneous deleterious materials on HMA performance parameters. However, because the amount and type of plastic fines in the fine aggregate can be used for predicting the performance of HMA pavements in terms of stripping (which may lead to permanent deformation), the amount and type of plastic fines were included in the research plan.

Toughness and Abrasion Resistance

The following general observations can be made from the literature review (Appendix A) concerning toughness and abrasion resistance:

- The Los Angeles abrasion test is the most widely used and investigated method for assessing aggregate toughness and abrasion resistance.
- The Los Angeles abrasion test is a dry test. Modifications to the standard Los Angeles abrasion test and differences with other ball mill type tests involve the introduction of water, (i.e., a wet abrasion process).
- Tests for aggregate toughness and abrasion resistance are closely related to soundness and durability.
- Correlations with performance are stronger for unbound base/subbase than for HMA.
- No comprehensive, definitive studies directly relate aggregate toughness and abrasion resistance properties with degradation during construction, HMA properties, or pavement performance.

Aggregate toughness and abrasion resistance could be used for predicting HMA performance in terms of resistance to degradation and weathering (which may lead to raveling, popouts, and potholing); therefore, these properties were included in the research plan.

Durability and Soundness

The literature review (Appendix A) indicates that soundness and freeze-thaw tests are most widely used for assessment of aggregate durability. Other tests, such as the durability index tests developed for western basalt aggregates, are specific to aggregates containing high-plasticity expansive clay minerals. In many of the studies, tests for toughness and abrasion resistance are combined with tests for durability and soundness. The link between aggregate soundness and the performance of base/subbase and portland cement concrete again appears stronger than the link with the performance of asphalt concrete pavement. The link between seal coat (chip seal or surface treatment) aggregate performance and the soundness test has also been established in some studies. This confirms speculation that individual aggregate particle expansion, shrinkage, or disintegration is more likely when uncoated and is more critical in a stiff, brittle matrix, such as portland cement concrete.

Aggregate durability and soundness affect the HMA performance in terms of raveling and popouts and, therefore, were included in the research plan.

Expansive Characteristics

Some aggregates, such as steel slag, tend to expand if not properly cured. The use of uncured or partially cured steel slag aggregate can cause significant swell and popouts in the HMA pavement and lead to its disintegration.

Although the expansive characteristics of an aggregate may cause distress in HMA pavements, it was not included in the research plan because it is not a general problem for aggregates and a reasonably adequate test method for quantifying this characteristic with relevant criterion is available.

Polish and Frictional Characteristics

Polish and frictional characteristics of the aggregate used in HMA wearing courses influence the friction between the HMA surface and vehicle tires. Frictional characteristics are determined by macrotexture and microtexture of HMA surface. Aggregate gradation controls macrotexture; basic rock properties control microtexture.

The locked wheel trailer is the primary method for measuring pavement surface friction properties. The British pendulum is the primary method for measuring aggregate or mix polish and friction properties in the laboratory.

The microtexture of coarse aggregate particles is the parameter most often used for assessing suitability of a rock source for providing frictional resistance. However, aggregate size is also known to influence pavement surface friction. In addition, other mix properties (e.g., asphalt content and air voids) affect pavement surface friction, which varies with the polishing action of traffic. Many HMA mixes contain aggregate from more than one source, which further complicates the delineation of the suitability of rock sources. This is often particularly true for fine aggregate components.

Because of the complexity of the problem, a test that measures only the microtexture of coarse aggregate may not be an efficient means of evaluating suitability for polish and friction resistance. A more efficient approach would seem to be to develop a test methodology for mixes and to include this testing as part of mix design; however, such an effort was considered beyond the scope of this study, so polish and frictional parameters were not included in this study.

Mineralogy and Petrography

The review of literature did not reveal strong relationships between mineralogy or petrology and the general performance of HMA pavements. This could be because all aggregate particles in HMA are coated with a film of asphalt binder; however, polish and friction is strongly linked to mineralogy and petrography. Studies have revealed relationships between aggregate mineralogy and polish/friction properties and have suggested that petrographic information be used with physical testing to establish criteria for aggregate polish and friction requirements. Stripping of HMA is also affected by mineralogy, but direct relationships have not been developed.

All aggregates used in this study were examined petrographically, and pertinent mineralogical properties were used as supplementary information in developing correlations between the physical properties of the aggregates and pavement performance parameters.

Chemical Properties

Chemical properties of aggregates have little effect on their suitability and performance in HMA, except as they affect adhesion of the asphalt binder to the aggregate and compatibility with antistripping additives that may be incorporated in asphalt binder. Asphalt binder must stick to the aggregate and resist stripping of the asphalt film in the presence of water. Therefore, the surface chemistry of the aggregate particles plays an important role in HMA performance. It was firmly established in SHRP studies that mineralogy and chemical composition of aggregate are of primary importance in stripping. There is abundant evidence that some aggregates have a greater affinity for water than for asphalt binder.

Aggregate chemical properties may have considerable influence on HMA pavement performance parameters such as permanent deformation and raveling (both resulting from the loss of adhesion between the aggregate and binder). The literature reviewed, however, does not offer promise that an implementable, performance-related test procedure or realistic chemical requirements can be selected or developed; therefore, aggregate chemical properties were not included in the research plan.

Properties of Minus 200 (P200) Material

Numerous studies (Appendix A) have shown that the properties of mineral filler (especially the material passing 75 μm or No. 200 sieve) have a significant effect on the performance of the HMA mixtures. Fines can influence the performance of HMA mixtures as follows:

1. Depending on the particle size, fines can act as a filler or as an extender of asphalt cement binder. In the latter case, an over-rich HMA mix can result, leading to flushing and/or rutting.
2. Some fines have a considerable effect on the asphalt cement—making it act as a much stiffer grade of asphalt cement than with the neat asphalt cement grade.
3. Some fines make the HMA mixtures susceptible to moisture-induced damage.

It is very important to characterize the fines so that the performance parameters of HMA pavements are not compromised; therefore, properties of P200 material were included in the research plan.

AGGREGATE PROPERTIES SELECTED FOR STUDY

Table 1 gives the HMA performance parameters and, on the basis of the literature review, those aggregate properties

TABLE 1 HMA performance parameters and pertinent aggregate properties

HMA Parameter	Aggregate Properties Which Have Major Influence
Permanent Deformation	(1) Coarse aggregate particle shape and surface texture (2) Fine aggregate particle shape and surface texture (3) Properties of P200 material[1] (4) Plastic fines in the fine aggregate[2]
Raveling, Popouts or Potholing[3]	(1) Toughness and abrasion resistance (2) Durability and soundness
Fatigue Cracking	(1) Coarse aggregate particle shape ans surface texture (2) Fine aggregate particle shape and surface texture (3) Properties of P200 material

[1]P200 material may cause permanent deformation of HMA by extending asphalt binder and/or inducing stripping in the HMA.
[2]Plastic fines in the fine aggregate may cause permanent deformation of HMA by inducing stripping in the HMA.
[3]These are HMA distresses which can result from the use of weak or unsound aggregates.

significantly influencing the performance parameters. The research work plan was based on aggregate properties because some influence more than one HMA performance parameter as is evident in Table 1. The following aggregate properties were included in the work plan:

- Coarse aggregate particle shape and surface texture,
- Fine aggregate particle shape and surface texture,
- Plastic fines in the fine aggregate,
- Toughness and abrasion resistance,
- Durability and soundness, and
- Properties of P200 material.

CHAPTER 3

SELECTION AND EVALUATION OF CURRENT AND POTENTIAL TESTS

IDENTIFICATION AND EVALUATION OF CURRENT AGGREGATE TEST PROCEDURES

To accomplish this task, the research team compiled the standard specifications for aggregates (and related test procedures) for HMA pavements used by the various states in the United States and foreign countries. A summary of aggregate tests and specifications was prepared and sent to the states for verification and updating (*5*). Letters were sent to other countries in North America, Europe, and Asia, and to Australia to obtain information on their aggregate tests and specifications for HMA construction. The responses were summarized. The aggregate tests most commonly used in the United States and other countries were identified from these two summaries. Appendix E contains a summarized state of the practice on aggregate tests and criteria for HMA in 1994. A discussion of these tests for each of the selected six aggregate test categories follows.

Particle Shape, Angularity, and Surface Texture (Coarse Aggregate)

The following test procedures (or variations thereof) are currently used in the United States and other countries to determine the particle shape and/or surface texture of coarse aggregates:

- ASTM D 3398 — Index of Aggregate Particle Shape and Texture
- ASTM D 4791 — Flat Particles, Elongated Particles, or Flat and Elongated Particles in Coarse Aggregate
- ASTM D 5821 — Percentage of Fractured Particles in Coarse Aggregate
- British Standard 812 (Part 105) Index of Aggregate — Flakiness Index of Aggregate British Standard 812 (Part 105) Elongation

Index of Aggregate Particle Shape and Surface Texture (ASTM D 3398)

Huang (*6*) developed a procedure for evaluating the particle shape and surface texture of coarse aggregates used in soil-aggregate road construction. The test was based on the concept that the volume of voids between packed, uniform-sized, coarse aggregate particles indicates the combined effect of shape, angularity, and surface texture of the aggregate. The results of the test were expressed as the particle index of the aggregate. This test was later revised by Huang (*7*) by modification of the void determination procedure. The test method later gained acceptance as a formal ASTM in which a series of three cylinders (203 mm [8 in.], 152 mm [6 in.], and 102 mm [4 in.] in diameter) were specified in lieu of the rhombohedron split mold used by Huang. McLeod and Davidson (*8*) conducted an extensive study and proposed the use of three molds (76 mm [3 in.], 51 mm [2 in.] and 38 mm [1.5 in.] in diameter) for testing fine aggregates using smaller sample sizes. The current ASTM uses five cylindrical molds ranging from 51 mm (2 in.) to 203 mm (8 in.) in diameter.

The equipment required for this test is simple, consisting basically of cylindrical steel molds of various diameters and corresponding steel tamping rods of different weights. The procedure requires that a clean, washed, oven-dried, one-sized aggregate fraction be used. The mold should be filled in three equal layers with each layer receiving ten tamps.

Each tamp consists of a drop with the tamping rod from 51 mm (2 in.) above the surface of the layer being compacted. The weight of the contents of the mold in each case is determined and the corresponding percentage of voids is calculated using the bulk specific gravity of each aggregate fraction. This procedure is repeated using the same material but applying 50 blows on each of the three layers. The particle index is then calculated using the following equation:

$$I_a = 1.25V_{10} - 0.25V_{50} - 32.0$$

where

I_a = particle index value

V_{10} = percent voids in the aggregate compacted with 10 blows per layer

V_{50} = percent voids in the aggregate compacted with 50 blows per layer

The particle index of an aggregate containing several size fractions is weighted on the basis of the percentage of the fractions in the original grading of the aggregate. An aggregate composed of rounded particles with smooth surface texture may have a low particle index of 6 or 7, while aggregates consisting of highly angular crushed particles with rough surface texture can have particle indexes of 15 to 20 or more.

This method, which requires separating different sieve fractions and testing each size, is time consuming and expensive (because of increased labor cost); therefore, it is not practical for routine testing of fine aggregates, which involve too many size fractions. It is suitable and practical for testing coarse aggregates, which have fewer size fractions to test. For example, AASHTO No. 8 coarse aggregate used in HMA surface courses would generally require testing of only one or two fractions. To date, no precision statement has been developed by ASTM for this test method. According to the survey, no state is using this test on a routine basis. Work done by Boutilier (*9*) and McLeod and Davidson (*8*) showed a fairly good relationship between the particle index and Marshall stability of HMA mixtures.

Flat Particles, Elongated Particles, or Flat and Elongated Particles in Coarse Aggregate (ASTM D 4791)

This method was adapted from U.S. Army Corps of Engineers Method CRD-C 119. It determines the percentages of flat or elongated particles in coarse aggregates, which are defined as those particles of aggregate having a ratio of width to thickness or length to width greater than a specified value. A proportional caliper device is used to identify the flat or elongated particles by testing individual particles of specific sieve sizes. The percentage of flat particles, elongated particles, and total flat and elongated particles is calculated either by number or by mass. This test is very simple. Its reproducibility is considered fair, although ASTM has yet to develop a precision statement.

Very little research has been conducted to correlate the percentages of flat, elongated, or flat and elongated particles to HMA pavement performance. Li and Kett (*10*) showed that the mix stability was adversely affected by the presence of flat aggregate particles. The HMA mix containing slabby-shaped aggregates had a significantly shorter fatigue life than the mix containing round gravel aggregate according to the study conducted by Maupin (*11*). However, Sharif (*12*) has indicated that a limited amount of flaky particles is beneficial for the fatigue behavior of HMA mixtures. Stephens and Sinha (*13*) reported that HMA mixes containing 30 percent or more flat particles maintained high void contents, which required increased asphalt contents, thereby resulting in low mix stability values. Livneh and Greenstein (*14*) reported that flaky aggregate particles tend to break more than cubical aggregate particles during the mix compaction process.

Very few states measure flat or elongated particles separately. Most states measure the ratio of the minimum dimension (thickness) to the maximum dimension (length) of the aggregate particle to determine the percentage of flat and elongated particles. Usually, a maximum percentage of ratios of 1:3 to 1:5 is specified. However, this ratio represents neither flatness nor elongation of the aggregate particles and, therefore, its engineering value is questionable.

The percentages of flat and/or elongated particles obviously are not a complete measure of aggregate particle shape. Moreover, the surface texture of the aggregate particles is not measured. Therefore, it is highly unlikely that these parameters by themselves can be a good indicator of HMA pavement performance. Most agencies use this test and related specification primarily to minimize degradation or breakage of aggregate particles during HMA production and compaction.

Percent of Fractured Particles in Coarse Aggregate (ASTM D 5821)

This test is conducted on the material retained on a No. 4 sieve. It is usually performed on gravel which need to be crushed to produce fractured face(s). A fractured face is a face that exposes the interior of the gravel particle. Fractured particles contained in a sample are weighed after separation and the percentage by weight determined. A face is considered a "fractured face" only if it has a projected area at least as large as one quarter of the maximum projected area of the particle and the face has sharp and well defined edges.

This test is primarily applicable to coarse gravel aggregate. It is practical and reasonable in cost. Because of the subjective determination of the fractured faces, its reproducibility is considered to be fair. The precision can be poor for materials with low fractured-particle counts (*15*). As reported in Appendix A, a significant amount of research has been done in relating the amount of fractured particles in gravel to the HMA mix stability. Most studies have shown that the mix stability increases as the percentage of fractured particles in gravel is increased. Most states using gravel aggregate in HMA mixtures specify minimum percentages of particles with fractured faces. This test does not quantify the aggregate particle shape nor its surface texture.

Some states specify the minimum percentage of particles with one or more fractured faces. Others specify the minimum percentage of particles with two or more fractured faces. The requirement varies for HMA base course, HMA binder course, HMA surface course, and OGFC.

Flakiness Index and Elongation Index (British Standard 812, Part 105)

The flakiness index test is used extensively in Europe and Australia. The flakiness index of an aggregate is the percentage by weight of particles in the aggregate whose least dimension (thickness) is less than 0.6 of the mean size. For this test, the mean size of the aggregate is defined as the mean of the two sieve aperture sizes between which the particle is retained by sieving. The test is not applicable to sizes smaller than 6.5 mm (1/4 in.). The aggregate is sieved through specified square aperture British Standard sieves. Each size fraction should give at least 200 pieces for testing. A metal thickness gauge with slotted openings is used to identify flaky particles. Each slot on the gauge comes identified with two sieve sizes between which the aggregate to be tested was retained. The width of the slot is 0.6 of the mean aggregate size. Slotted sieves of the appropriate slot width are also available for testing a large number of samples. The total amount of aggregate passing the gauge is weighed and is expressed as the percentage of the total sample to report the flakiness index.

The flakiness index method is not subjective and, therefore, does not depend on the tester's judgment. However a critical assessment of this test (*16*) indicated the following:

1. All three axes of the particles (not necessarily at right angles to one another) should be measured to obtain additional useful information about particle sphericity.
2. Caliper measurements are the most accurate means of assessing flakiness ratio and elongation ratio.
3. Judgment by eye was more accurate than the British Standard gauges.

Most other countries favor use of either caliper tests or a more accurate version of the British Standard gauges (*17*). The flakiness index method is believed to be reproducible, practical, and reasonable in cost.

The elongation index (British Standard 812, Part 105) is defined as the percentage by mass of the particles in a single-sized aggregate whose greatest dimension (length) is more than 1.8 times the mean dimension of the two sieves between which the aggregate to be tested was retained. Gauges with pins set with appropriate gaps are used to separate the particles by hand.

Particle Shape, Angularity, and Surface Texture (Fine Aggregate)

The following test procedures (or variations thereof) are used in the United States and other countries to determine the particle shape and surface texture of fine aggregates:

- ASTM D 3398 — Index of Aggregate Particle Shape and Texture
- AASHTO T 304 (ASTM C 1252) — Uncompacted Void Content of Fine Aggregate as Influenced by Particle Shape and Surface texture

Most state highway agencies control fine aggregate particle shape and texture in HMA mixtures by limiting the amount of natural sands, which are generally round in shape and smooth in texture. However, the use of generic terms such as *natural sand* or *manufactured sand* in specifications is not sufficient. Some natural sands are subangular rather than completely rounded. Similarly, not all manufactured sands are very angular (*18, 19*). There is a need to quantify the shape and texture of the fine aggregate in order to write appropriate specifications.

Index of Aggregate Particle Shape and Texture (ASTM D 3398)

This test has been described earlier for coarse aggregate. Several researchers (*7, 8, 9, 20, 21*) have shown a fairly good relationship between the particle index of fine aggregate and stability of HMA mixtures.

The test method requires testing each size fraction and, therefore, can be very time consuming and expensive because of increased labor cost. It is believed to be fairly reproducible, although the ASTM has not developed any precision statement. It is also possible to get meaningful test results by testing only one or two predominant size fractions, thus minimizing the testing time (*18*).

Uncompacted Voids

The uncompacted voids test has been adopted by the ASTM and is designated C 1252. It has been recommended by SHRP in the Superpave mix design system. AASHTO has now adopted this test as standard T 304.

In this method, a 100-cm^3 cylinder is filled with fine aggregate of prescribed gradation by allowing the sample to flow through a funnel into the calibrated cylinder. Excess material is struck off and the cylinder with aggregate is weighed. Uncompacted void content of the sample is then computed using this weight and the bulk dry specific gravity of the aggregate. Three variations of the method are being proposed. Method A uses a sample of specified gradation with material passing a 2.36 mm (No. 8) sieve and retained on a 150 μm (No. 100) sieve. Method B uses three individual size fractions: 2.36 to 1.18 mm (Nos. 8 to 16), 1.18 mm to 600 μm (Nos. 16 to 30) and 600 to 300 μm (Nos. 30 to 50); and the mean void content is determined. Method C uses the as-received gradation of all material passing a 4.75 mm (No. 4) sieve.

The performance predictability of this test is considered good based on the research work done at the NCAT (*2, 19*). It is a practical test, and its cost is also reasonable.

Plastic Fines in the Fine Aggregate

The following test procedures are used to ensure the cleanliness of aggregates and minimize the amounts of deleterious materials:

- AASHTO T 176 (ASTM D 2419) — Plastic Fines in Graded Aggregates and Soils by Use of the Sand Equivalent Test
- AASHTO T 90 — Determining the Plastic Limit and Plasticity Index of Soils (ASTM D 4318)
- AASHTO T 11 — Material Finer Than 75 μm (No. 200 Sieve) in Mineral Aggregate by Washing

Sand Equivalent Test

The sand equivalent test is used to determine the relative proportions of plastic fines or claylike material in fine aggregates. Fine aggregate passing the 4.75 mm (No. 4) sieve is placed in a graduated, transparent cylinder which is filled with a mixture of water and a flocculating agent. After agitation and 20 minutes of settling, the sand separates from the clay-like fines, and the heights of sand and sand plus clay are measured. The sand equivalent is the ratio of the height of the sand to the height of sand plus clay times 100. Higher sand equivalent will be obtained if a cleaner fine aggregate is used. Minimum specified sand equivalent values for fine aggregate in HMA range from 25 to 60 (*5*). The minimum requirement of 45 is most common. The requirement also depends on the type of HMA course, (e.g., base or surface course).

The sand equivalent test was developed by Hveem to control the quality of aggregates for HMA and untreated bases (*22*). Clough and Martinez (*23*) showed a very good correlation between sand equivalent value of the aggregate and the HMA mixture resistance to stripping. Also, according to Aschenbrener (*24*), the sand equivalent test value provides a reasonable indication of the HMA mix susceptibility to moisture-induced damage. This test is quick to perform; requires very simple equipment, which can be used with minimal training or experience; and has given reasonably good results.

Plasticity Index

The plasticity index (PI) is being used by several agencies to measure the degree of plasticity of fines. PI is the difference between the liquid limit and the plastic limit of the material passing a 425 μm (No. 40) sieve. ASTM D 1073 (Standard Specification for Fine Aggregate in Bituminous Paving Mixtures) and D 242 (Standard Specification for Mineral Filler for Bituminous Paving Mixtures) limit the PI of this fraction passing the 425 μm (No. 40) sieve (including the mineral filler) to a value of 4 or less. Some states specify a maximum PI for the P200 material. A review of literature indicates no reported correlation between the PI and the field performance of HMA. Precision data have not been established for liquid limit and plastic limit tests, which are based on the subjective judgment and experience of the tester.

Material Finer Than No. 200 Sieve by Washing

Material finer than the 75 μm (No. 200) sieve can be separated from larger particles much more efficiently and completely by wet sieving than through the use of dry sieving. Therefore, when accurate determination of material finer than 75 μm in fine or coarse aggregate is desired, this method is used on the sample prior to dry sieving (which may not be effective for some adherent fines or dust coatings). Clay particles and other aggregate particles that are dispersed by the wash water, as well as water-soluble materials, are removed from the aggregate during the test.

The test sample is placed in a container. Water (adding a dispersing agent to the water is optional) is poured into the container. The sample is agitated vigorously to bring the fine material into suspension. The wash water containing suspended and dissolved material is passed through a 75 μm sieve. The loss in weight resulting from the wash treatment is calculated as weight percent of the original sample and is reported as the percentage of material finer than a 75 μm sieve by washing.

This test method does not evaluate the quality of the P200 material—it measures the total quantity only; therefore, it is unlikely that the test values will be related to HMA pavement performance unless they are very excessive.

Toughness and Abrasion Resistance

The following test procedures (or variations thereof) are used in the United States and other countries to determine the toughness and resistance to abrasion of coarse and fine aggregates:

- AASHTO T96 (ASTM C 131) — Resistance to Degradation of Small-Sized Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine
- British Standard 812: Part 3 — Aggregate Impact Value (AIV)
- British Standard 812: Part 3 — Aggregate Crushing Value (ACV)

The survey of 45 states in the United States has indicated that 94 percent of the states surveyed use the Los Angeles abrasion test or some variation.

Resistance to Degradation of Small-Sized Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine (AASHTO T96-ASTM C131)

The Los Angeles abrasion test is the most widely specified test for evaluating the resistance of coarse aggregate to degradation by abrasion and impact. The test was originally developed in the mid-1920s by the Municipal Testing Laboratory of the city of Los Angeles, California. A 5,000-gm sample of aggregate having a specified grading is placed in a steel drum along with 6 to 12 steel balls each weighing about 420 gm. The drum is rotated for 500 revolutions. A shelf within the drum lifts and drops the aggregate and steel balls about 68 cm (27 in.) during each revolution. The resulting vigorous tumbling action combines impact, which causes the more brittle particles to shatter, with surface wear and abrasion as the particles rub against one another and against the steel balls. Following the completion of 500 revolutions, the sample is removed from the testing machine and sieved dry over a 1.70 mm (No. 12) sieve. The percent passing the 1.70 mm (No. 12) sieve, termed the percent wear or percent loss, is the Los Angeles degradation value for the sample.

Although widely used, the predictive capability of the Los Angeles abrasion test for coarse aggregate is rated only fair (the predictability for fine aggregate is unknown). Some developmental studies, for example, Woolf (*25*) and Melville (*26*), indicated good correlations with performance.

The reproducibility of the test for coarse aggregate can be rated as fair. The ASTM precision statement for the 19-mm (3⁄4-in.) nominal size grading allows a 12.7 percent multilaboratory difference and a 5.7 percent single-operator difference. The test is relatively simple and practical, and the cost of obtaining data is reasonable.

Aggregate Impact Value (British Standard 812)

In this test, a standard sample, with sizes ranging from 14.0 to 10.0 mm in diameter is subjected to a discontinuous loading in the form of 15 blows from a 100-mm-diameter hammer or piston (13.5 kg to 14.0 kg) falling 380 ± 5 mm. Non-standard aggregate sizes may also be tested. The sample suffers degradation to a graded assemblage of fines. A 2.36-mm sieve, which corresponds to approximately the U.S. Standard No. 8 sieve size, is chosen as the diagnostic cut-off level and the percentage of material passing, relative to initial weight, gives the aggregate impact value. The aggregate impact value provides a measure of resistance to granulation. Two tests are performed, and the results should be within a numerical value of 1. A lower numerical value indicates a tougher, more wear-resistant aggregate. Performance predictability is unknown although this is a standard test in Britain. Bullas and West (*27*) report the aggregate impact value did not separate suitable and unsuitable aggregate. Fookes, Gourley, and Ohikere (*28*) recommend that combination of physical tests (e.g., impact, crushing, and abrasion resistance) be used to assess aggregate durability. The reproducibility of results in the test is good, so that two tests per material have proven sufficient. The apparatus used is relative portable and cheap to operate, allowing both laboratory and field testing.

Aggregate Crushing Value (British Standard 812)

In this test, a sample of approximately 2 kg is subjected to a continuous compressive load transmitted through a piston. A total load of 400 kN on a 150-mm-diameter piston or a load of 100 kN on a 75-mm piston is achieved in 10 min. As in the aggregate impact value, the fines passing the British Standard 2.36-mm sieve are measured and a percentage of the initial sample weight calculated. This is the aggregate crushing value. Again, a lower value indicates a stronger and tougher aggregate. Bullas and West (*27*) report that crushing resistance did a good job of separating suitable and unsuitable aggregate. The reproducibility is rated as good. Two values are produced for each material and should be within a value of 1. The equipment and test procedure are simple and inexpensive, although a loading machine is required.

Durability and Soundness

The following test methods are generally used to determine the resistance of aggregates in HMA mixtures to degradation when exposed to weathering (i.e., wetting-drying and/or freezing-thawing):

- AASHTO T 104 (ASTM C 88) — Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate
- AASHTO T 103 Freezing and Thawing — Soundness of Aggregate by (3 Procedures)

Results of the survey of state specifications indicated that 54 percent of the states have a requirement for sodium sulfate soundness, 19 percent have a requirement for magnesium sulfate soundness, 9 percent have a freeze-thaw loss requirement, 2 percent require the durability index test, and 16 percent do not have a soundness requirement (*5*).

Durability and soundness tests are designed to simulate the destructive action of environmental factors (i.e., wetting-drying and freezing-thawing). Freezing-thawing are more detrimental than wetting or wetting-drying and, as a result, most test procedures simulate freezing-thawing. Water in pores or voids expands upon freezing, causing a breakdown of aggregate particles. The sulfate soundness tests were developed to simulate this action and were used in lieu of freezing and thawing because of the lack of adequate refrigeration equipment. Reliable, relatively inexpensive refrigeration equipment is now available, but the sulfate tests are still used extensively. The aggregate durability index test (AASHTO T 210) and similar tests (*29, 30, 31, 32*) were developed to detect harmful expansive clay minerals in weathered basalt indigenous to some western states.

Soundness of Aggregate by Sodium Sulfate or Magnesium Sulfate (AASHTO T 104 - ASTM C 88)

The sodium sulfate or magnesium sulfate test is widely used as an index of general aggregate quality. The soundness is intended to provide an estimate of the resistance of aggregate to weathering action. In the soundness test, the test sample is washed, dried and separated into specified size fractions. Each sample then is immersed in a solution of sodium or magnesium sulfate of specified strength for 16 to 18 hr at a temperature of 21 ± 1°C (70 ± 2°F). Next, the sample is removed from the solution and permitted to drain for 15 ± 5 min. It is then dried at a temperature of 110 ± 5°C (230 ± 9°F) until constant weight is achieved. Usually, the sample is subject to five cycles of immersion and drying. During the immersion cycle, the sulfate salt solution penetrates the pores in the aggregate. Partial or complete oven-drying dehydrates the sulfate salt precipitated in the pores. The internal expansive force, derived from the rehydration of the sulfate salt upon re-immersion, is intended to simulate the expansion of water upon freezing. After completion of the required number of immersion and drying cycles, the sulfate salt is washed out of the sample. The sample is sieved through specified sieves somewhat smaller than the original sieves on which a given size fraction was retained. The resulting weighted average loss for each size fraction is used as the indication of durability of the aggregate.

Some early studies, such as Paul (*33*), report good correlations of sulfate soundness test with performance while others, such as Garrity and Kriege (*34*), report poor correlations. Later studies—such as Gandhi and Lytton (*35*), Papaleontiou et al. (*36*), Hasan et al. (*37*), Rogers et al. (*38*) and Senior and Rogers (*39*)—also report mixed reviews for performance prediction.

Reproducibility of the sulfate tests is believed to be fair and poor for coarse and fine aggregate, respectively. The AASHTO (ASTM) procedures have a precision statement for coarse aggregate but not for fine aggregate. The 1S and D2S values are larger for sodium than for magnesium sulfate. Generally, these values appear large from an engineering perspective. In the literature reviewed, lack of precision was mentioned often as a problem with the test. Cost is rated as reasonable, even though the test is labor- and time-intensive.

Soundness of Aggregates by Freezing and Thawing (AASHTO T 103)

This test is designed to furnish information for judging the soundness of aggregate subjected to weathering. To perform the test, aggregate is washed and dried to constant mass at a temperature of 110 ± 5°C (230 ± 9°F) and then separated into individual size fractions by sieving. There are three procedures for immersion. In procedure A, samples are immersed in water for 24 hr prior to the start of freezing-thawing cycles. During freezing-thawing, the sample remains completely immersed. In procedure B, the samples are saturated by subjecting them to a vacuum of not more than 25.4 mm (1 in.) of mercury. Penetration into the aggregate pore is increased by using a 0.5 percent (by mass) solution of ethyl alcohol and water rather than water alone. After saturation, the sample is frozen and thawed in the alcohol-water solution. Procedure C is the same as procedure B except the water is used instead of the alcohol-water solution. Specifications may require 50, 16, and 25 cycles for procedures A, B, and C, respectively. After completion of the final cycle, samples are dried to constant weight and sieved. The resulting weighted average loss for each size fraction is used as the indication of soundness of the aggregate.

The performance predictability of AASHTO T 103 is unknown because it has not been used extensively. However, Senior and Rogers (*39*) rate the Ontario Canada version of the freeze-thaw test as superior to the sulfate soundness test. The Iowa DOT is satisfied with results from the alternate method that includes alcohol to aid penetration of pores. No precision statement is given in AASHTO T 103, and the reproducibility is unknown. Practicality and cost are rated as fair for reasons similar to the sulfate tests, although the necessary refrigeration equipment would increase the cost.

Properties of P200 Material

The following test procedures are most commonly used in the United States and other countries to test the material passing 75 μm (No. 200) sieve or P200 material:

• Plasticity Index (AASHTO T90) (ASTM D4318)	Determining the Plastic Limit and Plasticity Index of Soils
• Particle Size Distribution	Particle Size Analysis of Soils (AASHTO T 88) (Hydrometer method)
• Particle Size Distribution	Particle Size Analysis (Automated Laser Devices)

About 78 percent of states have some specification for mineral filler usually specifying gradation and/or plasticity (*5*).

Plasticity Index

Most highway agencies specify that the mineral filler shall comply with the requirements of AASHTO M 17 (Mineral Filler for Bituminous Paving Mixtures). AASHTO M 17 specifies a PI not greater than 4. However, the PI is applicable to the entire gradation of the mineral filler, which may contain as much as 30 percent material retained on a 75 μm (No. 200) sieve based on the following gradation specified in AASHTO M 17:

Sieve	*Percent Passing (by weight)*
600 μm (No. 30)	100
300 μm (No. 50)	95–100
75 μm (No. 200)	70–100

The PI limits have to be different for P200 material. Some agencies use a PI value of 5 when using the AASHTO M 17 gradation. PI limits are not suitable for hydrated lime and hydraulic cement.

PI test results and the amount of the P200 material have been used in judging the quality of the fine aggregates (*40*); however, a review of literature shows little research on the relationship of the PI value of the P200 material to the performance of HMA mixtures. Only a small percentage of HMA plants add mineral filler separately to the HMA mix. Most HMA plants have sufficient baghouse fines to meet the quantitative requirements of the P200 material. Therefore, there is a need to characterize the P200 material.

Particle Size Distribution

Some highway agencies specify the gradation of the P200 material. For example, Michigan specifies that the fraction passing the No. 200 sieve (75 μm) should not have more than 60 percent nor less than 15 percent finer than 10-μm diameter as determined in accordance with AASHTO T 88.

Minnesota specifies the following gradation based on AASHTO T 88 (using Gum Arabic as a dispersing agent if flocculation occurs):

Percent finer than 20 μm	35–100
Percent finer than 5 μm	10–40
Percent finer than 1 μm	1–25

Several studies (*24, 41, 42*) have shown that the gradation by itself does not explain the effect of the P200 material on the properties of the HMA mixtures—other characteristics of the P200 material must also be taken into account.

The FHWA has obtained a considerable amount of data on the gradation of the P200 material in their Demonstration Project 74. The FHWA uses a device that is much faster than the hydrometer analysis. Because the gradation of the P200 material in a HMA plant changes considerably during the production (*42*), the use of an automated device is desirable to conduct more tests.

Other test methods for characterizing the P200 material (e.g., the German filler test, Rigden void content, and methylene blue test) are used in some countries on a limited scale. These test methods will be discussed later.

IDENTIFICATION AND EVALUATION OF POTENTIAL TECHNIQUES FOR MEASURING AGGREGATE PROPERTIES

Several new techniques have been identified as having potential for evaluating the six selected aggregate test categories. Discussion of these techniques or tests follows.

Particle Shape, Angularity, and Surface Texture (coarse aggregate)

The following two techniques have been identified as having potential for quantifying the coarse aggregate particle shape, angularity, and surface texture:

- Particle Shape from Image Analysis Techniques
- Uncompacted Voids in Coarse Aggregate (similar to ASTM C1252 or AASHTO T 304 for fine aggregate)

Particle Shape from Image Analysis Techniques

Barksdale et al. (*43*) have used image analysis techniques to quantify the shape of several Georgia coarse aggregates. They used 12.7 mm to 9.5 mm (½ in. to ⅜ in.) fractions of the coarse aggregate. Fifty aggregate particles were placed flat in a small box which had a clear plastic bottom and dividers so as to give five rows of 10 aggregate particles each. A copy machine was used to provide an image of the 50 particles. The maximum length and average width of the particles were digitized directly from the photocopy using a digitizing tablet. Shadows were created when trying to photocopy the profile of the aggregate to measure the particle thickness. However, aggregate thickness was not digitized directly from the photocopy. Instead, the average thickness of the particles was measured with vernier calipers. The calipers, open to the proper width, were laid on the digitizing pad and the tips of the calipers, representing thickness of the particle, were digitized. A pen-type digitizer, rather than the one with cross-hairs, was used to digitize the vernier caliper measurements.

With experience, an operator can digitize the three dimensions of 150 aggregate particles in about 30 to 45 min according to the Georgia study (*43*). After digitizing the three dimensions, the data are saved as an AUTOCAD DXF file in ASCII code. Flatness ratio and elongation ratio can be calculated from the three dimensions, and typical shape classification (i.e., cube, rod, disc, and blade) scatter diagrams can also be plotted.

Two important test parameters are obtained from this imaging technique: sphericity and surface area. Sphericity is the ratio of the surface area of a sphere of the same volume as the particle to the surface area of the particle. When the sphericity value is close to 1.0, the particle is assumed to have a shape similar to a sphere. The surface area of each particle is obtained by the software developed at the Georgia Institute of Technology. Two aggregate sizes: −12.5 mm + 9.5 mm (−½ in. + ⅜ in.) and −9.5 mm + 4.75 mm (−⅜ in. + No. 4) should be tested when evaluating AASHTO No. 8 aggregate. The weighted average should be calculated based on the amount of each fraction in the gradation of the HMA.

Image analysis techniques are fairly reproducible because the measurements are not subjective and, therefore, are not on the basis of the tester's judgement. Once the technique is fully automated, it is expected to be practical and reasonable in cost.

Uncompacted Voids in Coarse Aggregate

This test (*44*) is similar to the uncompacted voids in fine aggregate test (ASTM C 1252 or AASHTO T 304). The test uses the appropriate gradation (−12.5 mm + 4.75 mm or −½ in. + No. 4) used in the HMA mixture. The coarse aggregate is dropped through a funnel into a container. Both the funnel and containers are proportionally larger than those used in ASTM C 1252.

Particle Shape, Angularity, and Surface Texture (Fine Aggregate)

The following potential technique was identified to quantify the particle shape and surface texture of the fine aggregate.

Particle Shape from Image Analysis

This automated technique was developed at the University of Arkansas for the FHWA (*45*). The fine aggregate is spread on a glass plate, and a high resolution videocamera is used to capture the image of each particle. Modern digital imaging hardware, image analysis techniques, and computerized analysis were used to quantify aggregate shape. Although several measures of shape were analyzed, EAPP (i.e., the ellipse-based area of the object divided by the perimeter squared) index and Roundness index were chosen as having the most potential for prediction of HMA performance in terms of rutting.

The EAPP index is a property independent of size that attempts to describe a particle's similarity to an ellipse. An EAPP index of 1.0 describes an elliptical particle. The Roundness index attempts to identify circular (assumed to be spherical) particles and is computed using the Hough transform (*45*). A Roundness index value less than 0.5 indicates elongated or flattened particles. A Roundness index value greater than 0.7 indicates bulkier particles. As the Roundness index approaches 1.0, a more circular (assumed to be spherical) particle is defined.

Plastic Fines in the Fine Aggregate

Although the sand equivalent test has the potential to predict the performance of HMA mixtures in terms of moisture susceptibility, this test primarily determines the quantity of claylike materials in aggregate. The test does not evaluate the quality of the claylike material, which may be equally detrimental to HMA mix resistance to moisture-induced damage. The following technique could be used to evaluate the quality of the claylike fines, if any, present in aggregate.

Methylene Blue Test

This French test method is recommended by the International Slurry Seal Association (ISSA) to quantify the amount of harmful clays of the smectite (montmorillinite) group, organic matter, and iron hydroxides present in fine aggregate. The test method, "Determination of Methylene Blue Adsorption Value (MBV) of Mineral Aggregate Fillers and Fines," is contained in *Technical Bulletin 145* of ISSA (*46*). The principle of the test is to add quantities of a standard aqueous solution of the dye (methylene blue) to a sample until adsorption of the dye ceases.

A representative sample of dry fine aggregate is screened through the No. 200 sieve. The portion of the sample passing the No. 200 sieve is tested for MBV. Ten grams of the sample are dispersed in 30 gm of distilled water in a beaker. One gram of methylene blue is dissolved in enough distilled water to produce 200 ml of solution, so that 1 ml of solution contains 5 milligrams of methylene blue. This methylene blue solution is titrated step wise in 0.5 ml aliquotes from the burrette into the continually stirred fine aggregate suspension. After each addition of methylene blue solution and stirring for 1 min, a small drop of the aggregate suspension is removed with a glass rod and placed on a filter paper. Successive additions of methylene blue solution are made until the end point is reached. Initially, a well-defined circle of methylene blue-stained dust is formed and is surrounded with an outer ring or corona of clear water. The end point is reached when a permanent light blue coloration or "halo" is observed in this ring of clear water. The methylene blue value (MBV) of a specific fine aggregate fraction is reported as milligrams of methylene blue per gram of specific fine aggregate fraction such as: MBV = 5.3 mg/gm,0/#200. The MBV expresses the quantity of methylene blue required to cover the total surface of the clay fraction of the sample with a mono-molecular layer of the methylene blue. Therefore, the MBV is proportional to the product of the clay content times the specific surface of the clay (*17*).

Results from the methylene blue test can be interpreted as a general rule-of-thumb as shown in Table 2 (*24*).

TABLE 2 Relationship of methylene blue values and anticipated pavement performance

Methylene Blue (mg/g)	Expected Performance
5 - 6	Excellent
10 - 12	Marginally acceptable
16 - 18	Problems or possible failure
20+	Failure

Both the methylene blue and sand equivalent tests appeared to provide comparable results in characterizing fine aggregates for moisture-susceptibility of HMA mixes in a Colorado Study (*24*). The methylene blue test is simple and practical, and its cost is reasonable. An Ohio DOT version of the test (Appendix D) was used in this study.

Toughness and Abrasion Resistance

The following two techniques have been identified as having potential to determine the toughness and abrasion resistance of aggregates.

Superpave Gyratory Compactor

A laboratory study was performed by Moavenzadeh and Goetz (*47*) using a gyratory testing machine to determine the factors affecting the degradation of aggregates in asphalt concrete. The gyratory testing machine was used to simulate the compaction of asphalt concrete under rollers and subsequently under traffic. This study showed a potential for the gyratory compactor to evaluate the toughness and abrasion resistance of aggregates through an interparticle abrasion and grinding action. In Los Angeles abrasion tests, the steel balls impart severe impacts on the aggregate particles, overshadowing interparticle abrasion which is probably the predominant process in pavement subjected to traffic loading. The Superpave gyratory compactor would be very suitable because it is available in most DOT laboratories for performing Superpave volumetric mix design for HMA mixtures. The Superpave gyratory compactor applies gyrations to the sample with a vertical pressure of 0.6 Mpa (87 psi), a 1.25-deg gyration angle, and at a rate of 30 gyrations per minute. The use of the gyratory compactor will permit testing of the entire mix gradation. This will mean that differences in degradation of dense-graded and open-graded (such as friction courses and SMA) HMA mixtures can be detected.

The number of gyrations can also be varied to better simulate the energy levels required to achieve compaction of HMA mixes for variable traffic. After compaction, gradation can be measured and compared to original gradations. Differences can be analyzed on the basis of particle size and surface area differences to evaluate degradation.

Micro-Deval Abrasion Test

This test, developed in France during the 1960s, is being used in the Province of Quebec and is being adopted by the Ontario Ministry of Transportation (*39*). The Los Angeles abrasion test, which is run dry, is not appropriate for some fine-grained, soft-rock aggregates (e.g., argillaceous carbonates and shales) which tend to absorb the impact energy of the steel balls and thus give low Los Angeles abrasion test values. However, these materials may be susceptible to slaking and particle degradation when wet, thereby giving poor pavement performance (*39*). It is not possible to test the aggregate in moist or wet condition in the Los Angeles abrasion machine because the fines tend to adhere to the side of the drum. The Micro-Deval test was developed to include the influence of moisture, which may significantly alter aggregate toughness and abrasion resistance.

For testing coarse aggregate in the Micro-Deval device, a 1,500-gm sample is initially soaked for at least 1 hr with 2 liters of tap water. The sample, water, and an abrasive charge of 5 kg of 9.5-mm-diameter steel balls are placed in a jar and revolved at 100 rpm for 2 hr. The sample is then washed and oven-dried. The loss is the amount of material passing the 1.18-mm (No. 16) sieve expressed as a percent by mass of the original sample. For coarse aggregate normally passing the 19.0 mm sieve, a 1,500-gm sample consists of 375 gm retained on the 16.0-mm sieve, 375 gm retained on the 13.2-mm sieve and 750 gm retained on the 9.5-mm sieve. In cases where the maximum nominal size is less than 16.0 mm, a 1,500-gm sample consists of 750 gm passing the 13.2 mm and retained on the 9.5 mm sieve, 375 gm retained on the 6.7-mm sieve and 375 gm retained on the 4.75-mm sieve. Comparison with the magnesium sulfate test indicates similarity of test results, but the Micro-Deval test has been found to be relatively insensitive to sample grading and also to have lower within-laboratory variation (about 1 to 2 percent). On the basis of an extensive study (which involved correlating test results with the field performance of HMA pavements), Senior and Rogers (*39*) have recommended the Micro-Deval test or aggregates to be used in HMA mixtures. The Micro-Deval test is a rapid, simple test—testing can be completed in 2 days. The equipment has been standardized and reference materials are also available from the Ontario Ministry of Transportation for periodic calibration of test equipment and procedures. Average losses for the reference material are 12.0 percent for the 19.0- to 9.5-mm grading and 14.5 percent for the 13.2- to 4.75-mm grading.

Durability and Soundness

The following two techniques were identified as having potential to evaluate the durability and soundness of aggregates.

Aggregate Durability Index (AASHTO T 210)

The durability index is a value indicating the relative resistance of an aggregate to produce detrimental claylike fines when subjected to prescribed mechanical agitation in the presence of water. Separate and different procedures are used to evaluate the coarse and the fine portions of the aggregate. The test assigns an empirical value to the relative amount, fineness, and character of claylike fines produced. The procedure has been used in limited geographical areas of the United States (primarily the West), and the results have been correlated with aggregate performance in construction applications primarily as unbound aggregate base. It is especially suitable for basalt-type aggregates containing interstitial montmorillonite. Research has indicated that it may be suitable for use instead of the sodium sulfate soundness test for evaluating the durability characteristics of fine aggregate for use in Portland cement concrete, thereby reducing the need for time-consuming and expensive soundness tests (AASHTO Materials, Part II, Tests, 1986).

Although no research data on the applicability of the aggregate durability index test to the aggregates used in HMA mixture exist, this test or a similar version of this test has been adopted by several states, and it is considered prudent to include it as a potential test for durability and soundness. However, the Micro-Deval test, discussed earlier, may provide comparable evaluations.

The aggregate durability index test for coarse aggregate can be summarized as follows. A washed and dried sample of coarse aggregate is agitated in a mechanical washing vessel for 10 min. A modified Tyler portable sieve shaker is used for agitation. The resulting wash water and -75 µm fines are collected and mixed with a stock calcium chloride solution and placed in a plastic cylinder. After a 20-min sedimentation time, the level of the sediment column is read. The height of the sediment is then used to calculate the durability index of the coarse aggregate.

A similar method is given for testing the aggregate durability index of fine aggregates. After the specified agitation period, the entire fine aggregate is tested for sand equivalent value (AASHTO T176) following a modified test procedure.

Canadian Freeze-Thaw Test

This test was developed at the University of Windsor and the Ontario Ministry of Transportation over a period of 10 years. It measures the resistance of aggregates to degradation when subjected to freezing and thawing in the presence of water containing deicing salts. Most highway agencies have depended on the sulfate soundness test as a simulation of the weathering environment. However, the crystallization of soluble salt in aggregate pores does not simulate the environmental conditions in North America. When the sulfate soundness test was developed, refrigeration equipment was not available in the laboratories. Now, equipment is readily available to simulate actual freezing and thawing conditions in the laboratory.

According to a Canadian study (*39*), the freeze-thaw test is marginally better than the magnesium sulfate test when used with petrographic examination for evaluating aggregates for HMA mixtures.

The Canadian freeze-thaw test is conducted by placing three fractions of aggregate into separate 1-liter jars. The aggregate size fractions are (a) 19.0 mm (3/4 in.) to 13.2 mm (1/2 in.), (b) 13.2 mm (1/2 in.) to 9.5 mm (3/8 in.), and (c) 9.5 mm (3/8 in.) to 4.75 mm (3/16 in.). The mass of the fractions are 1,250 gm, 1,000 gm, and 500 gm, respectively. Each fraction is soaked for 24 hr in a 3 percent NaCl solution, drained, sealed and cycled five times, frozen for 16 hr at $-18°$ C (0° F) and thawed at room temperature for 8 hr. The material then is drained, dried, and resieved using the retained sieve sizes, (i.e., 13.2 mm, 9.5 mm and 4.75 mm). The weighted average loss for the sample is determined from the "as received" grading and the percent loss from all three fractions. A reference material, with an average freeze-thaw loss of 24.5 percent, is used to calibrate test equipment and procedure.

Properties of P200 Material

The following four test methods were identified as having potential for characterizing the P200 material:

- Rigden Voids (British Standard 812)
- Rigden Voids (Penn State Modified)
- Methylene Blue Test
- German Filler Test

Rigden Voids

Void content in fines (generally called Rigden voids) compacted to maximum density appears to have good potential for characterizing the fines. Void content is regulated by four basic properties of fines—particle shape, particle size, particle-size distribution, and particle surface structure (*41*). A sample of vacuum-oven-dried fines is either vibrated in a graduated cylinder (*41*) or compacted in a small mold with a compaction hammer (*48, 49*) to maximum packing. Mass (gm) of the compacted fines is divided by the compacted volume (cm^3) to calculate bulk specific gravity (G_{fB}) of compacted fines. Apparent specific gravity (G_{fS}) of the fine solids is determined by AASHTO T 133 using kerosene. Void content in the fines compacted to maximum density is then calculated as follows:

$$V,\ percent = 100\left(1 - \frac{G_{fB}}{G_{fS}}\right)$$

Rigden (*50*) showed that the voids in the filler at its closest packing are of major importance for the behavior of filler-asphalt systems. The basic concept has also been discussed by Heukelom (*51*). If a filler is mixed with less asphalt cement than is required to fill its voids, a stiff, dry product is obtained. Overfilling the voids with asphalt cement, on the contrary, imparts a fluid character to the mixture. Rigden's concept maintains that a portion of the asphalt cement can be regarded as "fixed" in the inter- and intra-granular pores of the filler, while the remaining part is "free." Thus, the functional volume percentages of "solid" and "fluid" phase differ from the compositional volume percentages of filler granules and asphalt cement.

Both British Standard (British Standard 812) and Penn State modified equipment (*49*) have been used to determine Rigden voids. They are based on the same concept but use a different compactive effort.

Methylene Blue Test

The methylene blue test was described in detail in the test category "Plastic Fines in the Fine Aggregate." This test has the potential for identifying those P200 materials likely to be detrimental to the resistance of the HMA mixtures to moisture-induced damage.

German Filler Test

The German filler test is a measure of the amount of mineral filler required to absorb 15 gm of hydraulic oil. The hydraulic oil is put in a small bowl, then 45 gm of mineral filler is added and mixed. An attempt is made to form a ball with the mixture. If a ball is formed and holds together, more mineral filler, in 5-gm increments, is added. This process is continued until the mixture loses cohesion. At this point, all of the hydraulic oil is fixed in the voids of the P200 material and there is no excess to hold the particles together. The total amount of P200 added to the hydraulic oil is reported as the test value. This simple test procedure provides similar information about Rigden voids.

CHAPTER 4

LABORATORY TESTING AND RESULTS

RESEARCH PLAN

Selection of Materials

In selecting aggregate types for specific studies, the following general principles guided the selection process:

1. The selected aggregates should produce a wide range of test values for the specific property to be evaluated. For the particle shape and texture study, aggregates ranging from smooth rounded sand/gravel to angular rough textured slags should be selected. Aggregate for the plastic fines in the fine aggregate and P200 studies should be selected to give fines composed of dust of fracture from crushing to naturally occurring clay minerals. For the toughness and abrasion resistance and durability and soundness studies, performance history should be factored into the selection process. Where possible, both good and poor performing aggregates with a wide range of properties should be selected. Because of their extensive use in the United States, the Los Angeles abrasion test can provide the basis for selecting aggregate for the toughness and abrasion resistance study and the sulfate soundness test can be used as the basis for the durability and soundness study.
2. The selected aggregates should reflect a wide range of mineralogical composition (rock types). The volume of usage should also be factored into the selection process. As a result, carbonate stone should be weighted more heavily than other mineral types. A softer, lower, specific gravity high-calcium limestone and a hard, dense dolomite can be used in the particle shape and texture study. Several carbonate sources, including a limerock, should be included in both the toughness and abrasion resistance and durability and soundness studies. Blast furnace slag should be used in the particle shape and surface texture study because of the rough surface texture of blast furnace slag. However, steel slag should be used in the toughness and abrasion resistance and durability and soundness studies because of the greater use of steel slag.
3. For the toughness and abrasion resistance and durability and soundness studies, aggregate with historical performance ranging from good to poor should be selected. The selection process should rely heavily on the experiences of state highway agencies. Several state highway agencies should be contacted to identify aggregate sources. These include states with a wide range of climates as delineated by the four SHRP climatic regions.
4. Use common aggregate sources in as many different test categories as possible. This will minimize the number of characterization tests and will also provide a common link among different test categories and related pavement performance parameters.
5. When possible, aggregates that have been or are being used in controlled field pavement performance studies should be included. Field studies considered may include, but should not be limited to, the SHRP Materials Reference Library (MRL); the Minnesota test road; Westrack; and other state, regional, or national pavement testing facilities.
6. Aggregate types and sizes used in different HMA mix types (dense graded, gap graded, and open graded) and HMA courses (surface, binder, and base) should be included. A performance-based PG64-grade asphalt cement (equivalent to the most commonly used AC-20 viscosity-graded asphalt cement) should be used in HMA mixtures for validation testing.

Mix Validation Tests

The following discussions and guidelines were used in selection of the mix validation tests.

Permanent Deformation and Fatigue Cracking

In many cases, the validation of aggregate test methods has to be conducted by testing the HMA mixture for permanent deformation and/or fatigue cracking. Two primary HMA mix design methods, Marshall and Hveem, are used in the United States. Both of these empirical methods have been used to establish optimum asphalt contents of HMA mixes. Both test methods obtain arbitrary stability (resistance to deformation) values of the HMA mixtures. These stabilities are neither based on fundamental engineering properties nor have been validated in the field to predict HMA permanent deformation. The Marshall and Hveem test methods also do not indicate the HMA potential for fatigue cracking.

Various types of creep tests (e.g., unconfined/confined and static/dynamic loading) have been used by researchers for laboratory evaluation of HMA permanent deformation. However, no creep test has been adopted by AASHTO or ASTM nor has any creep test been validated in the field. HMA beam tests have been used for evaluating the fatigue life of HMA mixes in the laboratory; however, no satisfactory correlations have been developed between the laboratory beam fatigue data and fatigue cracking in the field. Recently, an asphalt aggregate mix analysis system (AAMAS) was developed to evaluate HMA for permanent deformation and fatigue cracking: however, the AAMAS has not been validated in the field.

SHRP has recently developed Superpave mix design and evaluation procedures after a 5-year, \$50 million study (*52*). The models for Superpave intermediate mix analysis (previously called Level 2) and complete mix analysis (previously called Level 3), which quantify permanent deformation and fatigue cracking of HMA mixes, have been reported to be validated. The permanent deformation component was validated by applying wheel track loads to HMA mixtures at the University of Nottingham. Although Superpave procedures have not been validated fully in the field, these procedures, which are based on sound engineering principles, represent the best technology currently available. However, one should not rely solely on Superpave mix evaluation procedures. The use of a Georgia loaded wheel test (GLWT) should be included as an additional validation test for permanent deformation. This device has been used extensively by the Georgia DOT since 1988 to evaluate the rutting behavior of HMA mixtures. A round-robin test program, in which six state highway departments participated, was sponsored by the FHWA in 1990 to evaluate the GLWT. The test results were encouraging. HMA beam specimens (125 mm wide by 300 mm long) are fabricated for testing by the GLWT. The beams are subjected to 445 Newton (100 lb) wheel load, 690 kPa (100 psi) contact pressure, and 40°C (104°F) test temperature. The rut depth is measured after 8,000 load applications.

The use of a triaxial compression test as a means (with and without asphalt binder) for determining the strength of the aggregates and the relationship of this strength to permanent deformation was also evaluated. This test does not apply dynamic loading (such as in Superpave mix analysis or the GLWT) and, therefore, does not produce the rutting phenomenon encountered in the field.

Superpave mix design procedures (previously called Level 1) should be followed in order to develop the volumetric design of HMA mixtures. After preparation, the HMA mixture should be subjected to short-term aging following AASHTO PP20 procedures.

The Superpave volumetric mix design method requires specimen compaction with a gyratory compactor in accordance with AASHTO TP 4. The design asphalt content is selected at 4 percent air voids at N_{design} (number of gyrations required for design). The value of N_{design} is selected from a table given in the Superpave mix design manual on the basis of the traffic level as expressed in terms of 80 kN equivalent single-axle loads (ESALs) expected on the pavement and the design 7-day maximum air temperature for the pavement site.

For this study, an intermediate design traffic level (10^7 80-kN ESALs) and the highest design 7-day maximum air temperature (43° to 45°C) should be used. This combination requires a $N_{design}=119$. If a high design traffic level ($>10^8$ ESALs) is chosen, it will reduce the design asphalt content and will make the HMA mixture unduly stiff and, therefore, mask the effect of aggregate characteristics (such as particle shape and surface texture) on the HMA properties. The highest air temperature range is proposed in order to maximize the number of gyrations required to evaluate the HMA mix's potential for rutting in a hot climate.

The optimum asphalt content for a specific gradation of the HMA mix should be established as follows in accordance with the Superpave volumetric mix design. Two specimens (150 mm in diameter by 50 mm high) each are compacted in the gyratory compactor at four different asphalt contents. The asphalt content which gives 4 percent air voids at N_{design} is selected as the optimum asphalt content.

The Superpave intermediate mix analysis procedures should be used to evaluate the HMA mix resistance to permanent deformation, fatigue cracking, or both. This level of mix design involves performance tests which measure fundamental properties and predict pavement performance. The tests required are the simple shear at constant height, frequency sweep at constant height, and indirect tensile strength to predict permanent deformation and fatigue cracking. These predictions are based on validated material properties which have been linked to pavement performance (*52*).

For permanent deformation, the simple shear at constant height and frequency sweep at constant height are conducted at the effective temperature for permanent deformation (T_{eff} [PD]). This effective temperature is calculated from the pavement temperatures for the project site. For this study, T_{eff} (PD) will be selected to simulate the hot climate in the United States.

For fatigue cracking, the frequency sweep at constant height, the simple shear at constant height, and the indirect tensile strength tests (50 mm per min loading rate) are conducted at the effective temperature for fatigue cracking (T_{eff}[FC]). This effective temperature is also calculated from the pavement temperatures for the paving project site. The T_{eff}(FC) should be selected to simulate the cold climate in the United States to accentuate the fatigue cracking. At least four compacted specimens are required to conduct 10 tests for Superpave intermediate mix analysis and evaluation for permanent deformation and fatigue cracking as shown in Table 3.

TABLE 3 Test requirements for Superpave intermediate mix analysis

Method of Test	Number of Tests	
	T_{eff} (PD)	T_{eff} (FC)
Simple shear at constant height	2*	2*
Frequency sweep at constant height	2*	2*
Indirect tensile strength (12.5 mm/min loading rate)		2

*Tests performed on same specimens.
T_{eff} (PD) = Effective temperature for permanent deformation.
T_{eff} (FC) = Effective temperature for fatigue cracking.

Superpave intermediate mix analysis gives average rut depth in millimeters and fatigue cracking in percent of area for the HMA mixtures after the design service life. It is not necessary to use Superpave complete mix analysis procedures intended for high traffic because (a) it will generally be used for about 5 percent of the total projects in a state; (b) it takes into account a 10-year block of historical temperatures, structural layer information, and environmental effects model, which are unnecessary in this comparative laboratory study; and (c) it is time consuming (tests are done at three temperatures).

Table 4 lists the Superpave mix design tests and equipment used in this study.

Moisture Susceptibility

Moisture-induced damage generally leads to permanent deformation and/or cracking (HMA performance parameters); the following test methods should be used to determine the susceptibility of the HMA mixtures to moisture-induced damage:

- AASHTO T 283 Resistance of Compacted Bituminous Mixture to Moisture-Induced Damage and
- Hamburg Wheel-Tracking Device (HWTD)

Of all the test methods available for evaluating moisture susceptibility, AASHTO T 283 (modified Lottman test) is the most widely used and its reliability is considered relatively better than numerous other test methods (*3*). Recent field studies by Aschenbrener (*24*) indicated that AASHTO T 283 could reasonably predict the stripping potential of Colorado aggregates. The moisture susceptibility of the HMA mixtures is quantified in terms of retained tensile strength after water conditioning.

The HWTD was used for additional validation. The device measures the combined effects of rutting and moisture damage by rolling a steel wheel back and forth across the surface of an HMA slab submerged in hot water. The HMA slab is compacted in the laboratory by a mechanically operated linear kneading compactor. The slab measures 320 mm in length, 260 mm in width, and 80 mm in thickness. The slab weighs about 15 kg and is compacted to a void content of 7 ± 1 percent. The slab is secured in a reusable steel container with plaster of Paris. The slab is submerged in water, which can be heated from 25°C (77°F) to 70°C (158°F), the standard testing temperature being 50°C (122°F).

Two slabs can be tested simultaneously with two reciprocating solid steel wheels (diameter 203.5 mm and width

TABLE 4 Superpave mix design tests and equipment used

Method or Procedure	Test Equipment	Relevant SHRP or AASHTO Test Designation
Gyratory Compaction	Superpave gyratory compactor	M-002, AASHTO TP 4
Short and long-term aging	Forced draft oven	M-007, AASHTO PP20
Frequency sweep at constant height	Superpave shear test device	M-003, P-005
Simple shear at constant height	Superpave shear test device	M-003, P-005
Indirect tensile strength	Indirect tensile device	M-005

47.0 mm) loaded to 710 N. The length of travel is 230 mm and the average speed is 1.1 km/hr resulting in 53±2 passes/min.

The testing duration is 20,000 cycles; deformation is recorded and plotted after each cycle. On the cycles versus deformation plot, two distinct lines are generally observed. The first line (rutting line) indicates rutting in the HMA unaffected by stripping. The second line (stripping line) with a steeper slope indicates rutting because of stripping. The point (number of cycles) where the slope of the rutting line and the slope of the stripping line intersect is called the inflection point. This is the point where stripping is assumed to have been initiated. The inflection point (expressed in terms of number of cycles) is the test parameter of interest for this study.

Design of Experiments and Statistical Analysis

Both current and new test procedures were used to test all selected aggregates for a specific aggregate test property. All tests were conducted with three replicates (n=3) (except the Superpave intermediate mix analysis which used two replicates), and the results were averaged. As an example, if the number of aggregate types is 9, the number of aggregate test types and mix validation test types is 8, and the number of replicates is 3, the total number of tests to be conducted will be 9 by 8 by 3 = 216.

Single-variable correlation analysis and multiple regression analyses were made using aggregate test values as independent variables and mix validation test values as dependent variables. Simple correlations were also made between test results from different aggregate test types. Correlation matrixes were developed using the SAS computer program so that the relationship between different aggregate tests and the relationships between aggregate tests and HMA performance parameters could be examined in a tabular form. Correlation coefficients, (i.e., R values) were used in the correlation matrixes tables because their sign (+ or −) indicates the relationship between the variables. When linear equations ($y = a + bx$) are shown, coefficients of determination, (i.e., R^2 values) were used because the sign of the "b" coefficient indicates the relationship between the independent (x) and the dependent (y) variables.

The forward selection multiple variables procedure given in the SAS program was used to select the aggregate test which is related to HMA performance parameter. The forward selection procedure begins by finding the variable that produces the optimum one-variable subset (i.e., the variable with the largest coefficient of determination or R^2). In the second step, the procedure finds that variable which, when added to the already chosen variable, results in the largest increase in R^2 and so on. The process continues until no variable considered for addition to the model provides an increase in R^2 considered statistically significant at the specified level ($P = 0.05$ for this study).

FINDINGS OF COARSE AGGREGATE PARTICLE SHAPE, ANGULARITY, AND SURFACE TEXTURE STUDY

This segment of the study was designed to identify coarse aggregate test methods that best characterize aggregate shape, angularity, and/or surface texture and are related to permanent deformation and fatigue cracking of HMA pavements.

Aggregate Types and Test Results

Nine coarse aggregates were selected to provide a wide range of particle shape, angularity, and surface texture. Table 5 lists the aggregates. The following nine tests were chosen to characterize particle shape, angularity, and/or texture of the coarse aggregates:

TABLE 5 Coarse aggregates description and location

Sample Number	Description	Location
CA-1	Round Natural Gravel	Alabama
CA-2	Crushed Gravel-Small Feed	Alabama
CA-3	Crushed Gravel-Large Feed	Pennsylvania
CA-4	Sandstone	Alabama
CA-5	Limestone	Alabama
CA-6	Dolomite	Alabama
CA-7	Granite	Georgia
CA-8	Siltstone	Virginia
CA-9	Blast Furnace Slag	Alabama

- Index of Aggregate Particle Shape and Texture (ASTM D 3398)
- Image Analysis (developed by the Georgia Institute of Technology)
- Flat or Elongated Particles in Coarse Aggregate (ASTM D 4791)
- Flat and Elongated Particles in Coarse Aggregate (ASTM D 4791)
- Flakiness Index (British Standard 812)
- Elongation Index (British Standard 812)
- Percent of Fractured Particles in Coarse Aggregate (ASTM D 5821)
- Uncompacted Voids in Coarse Aggregate-Funnel Technique (similar to ASTM C 1252)
- Uncompacted Voids in Coarse Aggregate-Shovel Technique (after AASHTO T 19)

Only the flat and elongated particles and percent of fractured particles tests are routinely used in the United States. Tests for specific gravity and absorption of coarse aggregate (AASHTO T 85) and adherent fines (ASTM D 5711) were conducted for mix design and information only. Aggregate test results are shown in Table 6. Values shown are the averages of three replicates. The index of aggregate particle shape and texture (ASTM D 3398) was performed on each individual size fraction of the coarse aggregate, then the weighted average was calculated based on the amount of each size fraction in the gradation of the HMA tested. In the table, this weighted average has been called the combined index of particle shape.

Two parameters—sphericity and surface area—were obtained from the image analysis of coarse aggregate particles as mentioned earlier. Two sizes, −12.5 mm to +9.5 mm (−½ in. to +⅜ in.) and −9.5 mm to +4.75 mm (−⅜ in. to + No. 4), of the coarse aggregate were tested, and the weighted average was calculated on the basis of the amount of each size fraction in the gradation of the HMA tested.

Flat or elongated particles at 2:1, 3:1, and 5:1 ratios and flat and elongated particles (ASTM 4791) at 3:1 and 5:1 ratios were determined. The British tests for flakiness index and elongation index (BS 812) were conducted on specially prepared gradations, −14 mm to +10 mm and −10 mm to +6.3 mm as required in the standard. All measures of particle shape were weighted with respect to the amount of each size fraction used in the mix.

The test for the percentage of fractured particles (ASTM D 5821) was determined only on the gravels (CA-1 to CA-3). The only size fraction large enough to test by this method was the −12.5 mm to +9.5 mm (−½ in. to +⅜ in.) material.

A test similar to the uncompacted voids of the fine aggregate (ASTM C 1252) was performed. This test used the appropriate gradation (−12.5 mm to +4.75 mm [−½ in. to + No. 4]) of the coarse aggregate used in the mix. A sample size of 5,000 gm, consisting of 1,970 gm of −12.5 mm + 9.5 mm (−½ in. + ⅜ in.) and 3,030 gm of −9.5 mm + 4.75 mm (−⅜ in. + No. 4), was used.

The material is dropped through a funnel into a container. The funnel and container are proportionally larger than those used in ASTM C 1252. Three aggregates—a low, uncompacted voids; a high, uncompacted voids; and a mid-range voids—were tested with the shoveling technique (AASHTO T-19) to determine if the methods yielded relatively similar results. Although the results were not identical (see Table 6), they were consistent. The shoveling technique gave values approximately 2 percent less than the funnel technique for all three sources tested.

Mix Composition and Validation Tests

Figure 1 shows the gradation used for all HMA mixtures. To maximize the effect of particle shape, angularity, and surface texture on mix properties, each source was used exclusively for the −12.5 mm to +2.38 mm (−½ in. to + No.8) size fraction. The selected gradation was also below the Superpave restricted zone to maximize the amount and effect of coarse aggregate. The material passing 2.38 mm (− No. 8) was held constant for each mix. A natural rounded sand with an uncompacted voids (ASTM C1252) value of 40.9 and a Superpave PG 64-22 grade asphalt cement were used for all HMA mixes.

Superpave volumetric mix design was used to select the optimum asphalt content of all HMA mixtures. Two specimens (150-mm diameter by 115-mm height) were compacted, using the Pine gyratory compactor, at each of four asphalt contents after short-term aging at 135°C (275°F) for 4 hr. The asphalt content that gave 4 percent air voids at N_{design} (119 gyrations, for intermediate design traffic level of 10^7 EASLs) was selected as the optimum asphalt content. Results from the mix designs for all nine coarse aggregate sources are shown in Table 7.

Mixture validation tests using the Superpave shear tester (SST) and the indirect tensile tester (IDT) were conducted by the Asphalt Institute. Tests with the SST included simple shear at constant height and frequency sweep at constant height. Tests were run at 40°C and 20°C to compute parameters for evaluating rutting and fatigue cracking propensity, respectively.

The following three individual test parameters, which are used in Superpave intermediate mix evaluation, were computed to evaluate the propensity of the HMA mixtures for permanent deformation (rutting) and fatigue cracking:

1. *$G^*/\sin\delta$ at 0.1 Hz.*
   $G^*/\sin\delta$ of the HMA mix is similar to $G^*/\sin\delta$ (rutting parameter) of PG-graded asphalt binder. It is a measure of HMA stiffness at high pavement temperature (40°C) at a slow rate of loading (0.1 cycle/sec). Higher values of $G^*/\sin\delta$ indicate increased stiffness of HMA mixtures and, therefore, increased resistance to rutting. G^*

TABLE 6 Coarse aggregate test results

Mix #	CA-1	CA-2	CA-3	CA-4	CA-5	CA-6	CA-7	CA-8	CA-9
Test	Round Natural Gravel	Crushed Gravel-Small Feed	Crushed Gravel-Large Feed	Sand-stone	Lime-stone	Dolomite	Granite	Silt-stone	Blast Furnace Slag
Combined Index of Particle Shape ½"-#8	8.7	9.4	13.2	13.1	16.8	15.4	15.8	17.1	14.9
Sphericity (Image Analysis)	0.90	0.88	0.86	0.92	0.87	0.88	0.87	0.88	0.92
Surface Area (Image Analysis)	0.46	0.41	0.38	0.50	0.49	0.38	0.37	0.37	0.51
Flat or Elongated 2:1	21	20	39	22	46	45	57	49	11
Flat or Elongated 3:1	2	2	15	4	17	20	11	13	0
Flat or Elongated 5:1	0	0	2	0	4	12	1	3	0
Flat and Elongated 3:1	18	5	37	16	25	30	43	35	10
Flat and Elongated 5:1	0	0	9	2	6	4	2	6	1
Flakiness Index 14 mm-6.3 mm	16	14	30	16	28	19	45	29	3
Elongation Index 14 mm-6.3 mm	33	18	35	34	34	70	10	44	2
% Crushed in Gravel	16	77	92	---	---	---	---	---	---
Uncompacted Voids (Funnel Tech.)	43.1	44.5	47.4	48.0	50.5	50.1	49.5	52.2	50.8
Uncompacted Voids (Shovel Tech.)	40.9	--	45.7	--	48.1	--	--	--	--
Apparent SG	2.647	2.660	2.680	2.633	2.786	2.846	2.774	2.743	2.473
Bulk SG	2.608	2.619	2.556	2.508	2.763	2.829	2.744	2.690	2.260
Water Absorp.	0.6	0.6	1.8	1.9	0.3	0.2	0.4	0.7	3.8
Adherent Fines (%)	0.044	0.052	0.645	0.249	0.118	0.123	0.117	0.320	0.363

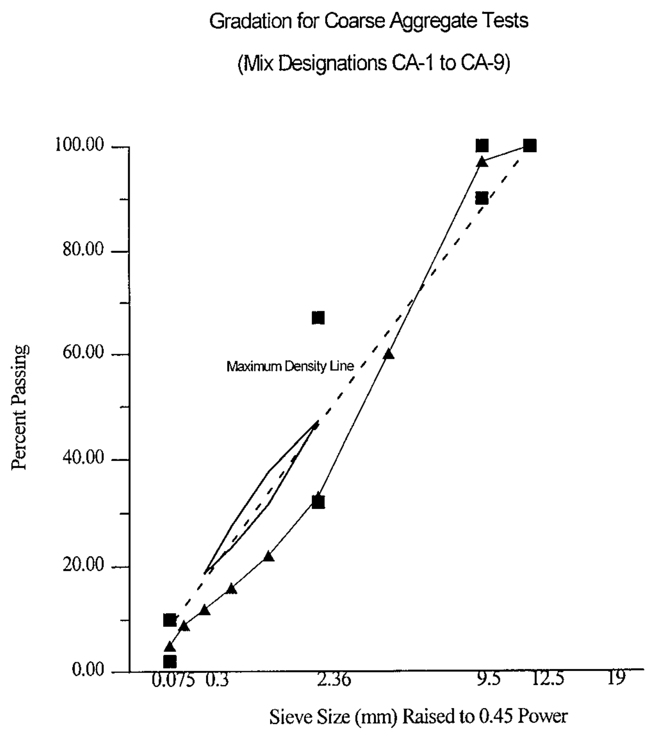


Figure 1. Gradation of HMA mixtures (coarse aggregate study).

is the complex modulus and δ is the phase angle when HMA is tested under dynamic loading.

2. *Slope (m) of the frequency versus G* plot.*
   The m value was obtained from the frequency sweep at constant height conducted by the SST at high effective temperature for permanent deformation or T_{eff}(PD) with frequencies ranging from 0.01 Hz to 10 Hz. In other words, G* (stiffness) of the compacted HMA specimen is measured at different frequencies. The slope (m) of the best fit line on the frequency versus G* plot is calculated. This slope represents the rate of development of rutting for the tested mix and is used in the Superpave model as such. The lower the m slope, the better is the mix's resistance to rutting.
3. *G*sinδ at 1.0 Hz.*
   G*sinδ of the HMA mix is similar to G*sinδ (fatigue factor) of the asphalt binder. It is a measure of the stiffness at intermediate effective pavement temperatures for fatigue cracking or T_{eff}(FC). G*sinδ was measured at 1.0 Hz to represent fast-moving traffic. A T_{eff}(FC) of 20°C was used. High values of G*sinδ at 1.0 Hz indicate high stiffness at intermediate temperatures and, therefore, low resistance to fatigue cracking according to Superpave.

Slab specimens were compacted and tested in the GLWT at the Georgia DOT. Table 8 contains the average voids in the total mix (VTM) and VMA data for the slab specimens. Slab preparation is controlled by compacting a required mass of mix in a mold of specified volume. The VTM values achieved were higher than the target value of 7 percent. Tests were conducted at 50°C. Each specimen was subjected to 8,000 wheel loads of 45 kg (100 lb) through hoses pressurized to 690 kPa (100 psi). Rut depths were manually measured at 1,000 cycle intervals.

The static triaxial compression test was explored as a third mix validation test. Two extreme HMA mixes containing CA-1 (round natural gravel) and CA-5 (limestone) aggregates were tested to determine the angle of internal friction ϕ which was expected to be related to the aggregate particle shape. High ϕ values are normally associated with angular aggregates. However, opposite results were noted. The ϕ values of CA-1 and CA-5 mixes were 47 deg and 39 deg, respectively. The higher ϕ value of CA-1 probably resulted from denser packing of round aggregate (VMA = 12.8) compared to CA-5 (VMA = 15.2).

Test results from the SST and GLWT are shown in Table 9. Rut depths for the GLWT are after 4,000 repetitions and not 8,000 repetitions because all values were not available after 8,000 repetitions. Values shown are averages of two replicates.

Statistical Analysis

Correlations Between Aggregate Properties

A correlation matrix was developed for all aggregate properties using correlation analysis. This was done to find out which test methods better characterize coarse aggregate particle shape and texture. Table 10 contains descriptions of each variable in the correlation matrix and Table 11 shows the correlation matrix itself. The top values in each cell are the correlation coefficients, R, and the bottom values are the significance levels, P. Higher absolute values of R imply

TABLE 7 Mix data for SST tests (coarse aggregate study)

Mix	Optimum Asphalt Content	Theoretical Maximum Density at Optimum	Average VTM	Average VMA
CA-1	4.1	2.503	3.9	12.8
CA-2	4.5	2.487	3.9	14.0
CA-3	5.6	2.425	3.7	15.5
CA-4	5.7	2.436	3.3	13.8
CA-5	4.75	2.544	4.0	15.2
CA-6	4.2	2.613	3.5	13.5
CA-7	5.25	2.512	4.0	16.2
CA-8	6.9	2.442	4.0	19.2
CA-9	8.5	2.306	3.9	15.3

TABLE 8 Mix data for GLWT slab specimens (coarse aggregate)

Mix	Average VTM	Average VMA
CA-1	9.7	18.1
CA-2	9.1	18.6
CA-3	10.0	21.0
CA-4	10.0	19.8
CA-5	11.3	21.6
CA-6	10.0	19.3
CA-7	9.8	21.6
CA-8	10.5	24.7
CA-9	11.2	21.8

TABLE 9 Coarse aggregate mix validation test results

Mix	Rutting Parameters			Fatigue Parameter
	Temperature 50°C	Temperature 40°C		Temperature 20°C
	GLWT Rut Depth (mm)	G*/sinδ @ 0.1 hz psi	m	G*sinδ @ 1.0 hz psi
CA-1	8.82	7126	0.4759	102131
CA-2	8.30	6460	0.4565	94850
CA-3	7.34	8136	0.4238	88843
CA-4	6.48	9052	0.3954	69632
CA-5	6.87	8643	0.4133	77600
CA-6	5.10	10962	0.4395	108296
CA-7	6.35	5870	0.4519	75509
CA-8	8.23	9797	0.3596	64176
CA-9	4.41	5347	0.4301	63386

stronger correlations and lower values of P imply greater significance.

Uncompacted voids (UV) has an excellent correlation ($R=0.97$, $P=0.0001$) with the index of particle shape (ASTM D 3398) but no significant ($P<0.05$) correlation with other particle shape tests. The index of particle shape has a significant and best correlation with uncompacted voids, followed by a significant and a fair correlation ($R=0.66$, $P=0.053$) with flat or elongated particles (2:1 ratio). Sphericity measured by imaging techniques has a good correlation with the flakiness index ($R=-0.77$, $P=0.015$) and flat or elongated particles (2:1 ratio) ($R=-0.76$, $P=0.018$). Sphericity decreases as the percentage of flat or elongated particles increases.

Aggregate tests quantifying flat (or flaky) and/or elongated particles are related to each other. Some relationships, significant at 95 percent level ($P<0.05$), are shown in Table 12.

TABLE 10 Description of variables for coarse aggregate properties

Variable	Description
Index	index of particle shape and texture
Sphericity	sphericity as measured by image analysis
SA	surface area as measured by image analysis
FOE21	% flat or elongated, ratio of 2:1
FOE31	% flat or elongated, ratio of 3:1
FOE51	% flat or elongated, ratio of 5:1
FI	flakiness index as measured by BS 812
EI	elongation index as measured by BS 812
UV	Uncompacted voids (funnel technique)
FAE31	% flat and elongated, ratio of 3:1
FAE51	% flat and elongated, ratio of 5:1

TABLE 11 Correlation matrix between coarse aggregate properties

	Index	Sphericity	SA	FOE21	FOE31	FOE51	FI	EI	UV	FAE31	FAE51
Index	1.0	-0.27 0.48	-0.15 0.70	0.66 0.05	0.63 0.07	0.44 0.24	0.45 0.23	0.27 0.49	0.97 0.0001	0.57 0.10	0.51 0.16
Spher-icity		1.0	0.75 0.02	-0.76 0.02	-0.68 0.04	-0.29 0.46	-0.77 0.02	0.05 0.90	-0.10 0.80	-0.64 0.06	-0.59 0.09
SA			1.0	-0.67 0.05	-0.52 0.15	-0.37 0.33	-0.64 0.06	-0.14 0.73	-0.07 0.85	-0.68 0.04	-0.32 0.41
FOE21				1.0	0.84 0.005	0.49 0.18	0.91 0.0006	0.19 0.63	0.50 0.17	0.90 0.0009	0.57 0.11
FOE31					1.0	0.79 0.01	0.63 0.07	0.58 0.10	0.51 0.16	0.75 0.02	0.78 0.01
FOE51						1.0	0.14 0.72	0.86 0.003	0.41 0.27	0.38 0.32	0.39 0.31
FI							1.0	-0.15 0.70	0.26 0.49	0.88 0.002	0.50 0.17
EI								1.0	0.32 0.41	0.19 0.63	0.39 0.30
UV									1.0	0.45 0.22	0.44 0.24
FAE31										1.0	0.63 0.07
FAE51											1.0

[a] Top values are correlation coefficients R and bottom values are significance levels P in each cell.

Table 12 shows the expected relationships between measures of flakiness (low ratios and flakiness index) and measures of elongation (high ratios and elongation index). There are poor correlations between FOE51 and FAE51 and EI and FAE51 as shown in Table 11.

Correlations Between Aggregate Tests and Mix Properties

A correlation matrix was developed between 11 aggregate test parameters and 4 mix validation parameters. Table 13 contains descriptions of each mix property variable. The correlation matrix (Table 14) shows that the correlations are generally poor. Various plots were examined to identify possible outliers. It was established that HMA mixes containing CA-6, CA-8, and CA-9 coarse aggregates had unusual mix properties and were, therefore, omitted from further analysis. CA-6 (dolomite) had unusually low asphalt content (4.2 percent) and relatively low VMA (13.5 percent), considering crushed coarse aggregate, which made equal comparison with other mixes difficult. CA-8 (siltstone) had very high VMA (19.2 percent) which resulted in a high asphalt content

TABLE 12 Significant correlations between measures of flat and/or elongated particle shapes

Parameters	R	P
Flat or Elongated 2:1 vs. Flat or Elongated 3:1	0.84	0.005
Flat or Elongated 2:1 vs. Flakiness Index	0.91	0.0006
Flat or Elongated 2:1 vs. Flat and Elongated 3:1	0.91	0.0009
Flat or Elongated 3:1 vs. Flat or Elongated 5:1	0.79	0.011
Flat or Elongated 3:1 vs. Flat and Elongated 3:1	0.75	0.021
Flat or Elongated 3:1 vs. Flat and Elongated 5:1	0.78	0.012
Flat or Elongated 5:1 vs. Elongation Index	0.86	0.003
Flakiness Index vs. Flat and Elongated 3:1	0.88	0.002

TABLE 13 Description of variables for mix rutting and fatigue properties

Variable	Description
RUT01	G*/sinδ at 0.1 hertz
RUTM	the slope of the linear regression line of the log G* vs. log Frequency curve at high temp
FAT10	G*sinδ at 1.0 hertz
GARUT	rutting as measured by the Georgia loaded wheel tester at 4000 passes

(6.9 percent) compared with other mixes. This aggregate, Virginia siltstone, has a history of unexplained bleeding and rutting in the field. CA-9 was a blast furnace slag mix, which had a very high asphalt content because of the absorptive nature and very rough surface texture of the porous slag aggregate.

When these three aggregates are deleted from the analysis because of the preceding assignable causes, the correlations improve significantly, as shown in Table 15.

Index of particle shape and texture (Index) and UV have relatively strong correlations with both rutting in the GLWT and the fatigue parameter G*sinδ at 1.0 Hz (FAT10). As shown in Table 15, three of the four correlations are significant at the 5 percent level ($P <= 0.05$) and the fourth, index of particle shape and texture with G*sinδ at 1.0 Hz, is close—with $P=0.06$. None of the other aggregate properties has strong correlations with any of the rutting or fatigue parameters.

TABLE 14 Correlation matrix between coarse aggregate properties and mix properties (all sources)

Aggregate Properties	Rutting Parameters			Fatigue Parameter
	GARUT	RUT01	RUTM	FAT10
Index	-0.492 0.18	0.338 0.37	-0.640 0.06	-0.540 0.13
Spher-icity	-0.334 0.38	-0.162 0.68	0.004 0.99	-0.250 0.52
SA	-0.266 0.49	-0.231 0.55	-0.012 0.98	-0.317 0.41
FOE21	0.036 0.93	0.412 0.27	-0.272 0.48	-0.008 0.98
FOE31	-0.126 0.74	0.702 0.03	-0.322 0.40	0.205 0.60
FOE51	-0.372 0.032	0.750 0.02	-0.101 0.79	0.456 0.22
FI	0.240 0.53	0.163 0.68	-0.172 0.66	-0.079 0.84
EI	-0.263 0.49	0.847 0.004	-0.268 0.49	0.373 0.32
UV	-0.571 0.11	0.326 0.39	-0.697 0.04	-0.607 0.08
FAE31	-0.019 0.96	0.325 0.39	-0.251 0.51	-0.052 0.89
FAE51	0.036 0.93	0.544 0.13	-0.536 0.14	-0.074 0.85

[a] Top values are correlation coefficients R and bottom values are significance levels P in each cell.

TABLE 15 Correlation matrix between coarse aggregate properties and mix properties (six sources)

Aggregate Properties	Rutting Parameters			Fatigue Parameter
	GARUT	RUT01	RUTM	FAT10
Index	-0.864 0.03	0.229 0.66	-0.572 0.24	-0.793 0.06
Spher-icity	0.101 0.85	0.386 0.45	-0.162 0.76	-0.100 0.85
SA	-0.036 0.95	0.741 0.09	-0.484 0.33	-0.252 0.63
FOE21	-0.652 0.16	-0.210 0.69	-0.119 0.82	-0.480 0.34
FOE31	-0.555 0.25	0.284 0.59	-0.450 0.37	-0.409 0.42
FOE51	-0.433 0.39	0.433 0.39	-0.486 0.33	-0.358 0.49
FI	-0.632 0.18	-0.300 0.56	-0.047 0.93	-0.436 0.39
EI	0.108 0.84	0.856 0.03	-0.461 0.36	0.050 0.92
UV	-0.916 0.01	0.321 0.54	-0.677 0.14	-0.863 0.03
FAE31	-0.572 0.24	-0.137 0.80	-0.097 0.85	-0.359 0.49
FAE51	-0.364 0.48	0.529 0.28	-0.558 0.25	-0.235 0.65

[a] Top values are correlation coefficients R and bottom values are significance levels P in each cell.

TABLE 16 Coarse aggregate particle shape multiple regression equations

Step	Dependent Variables	Independent Variables	Equation	R^2	P
1	GARUT	UV	GARUT=22.52-0.32(UV)	0.84	0.0104
2	GARUT	UV, FOE51	GARUT=28.82+0.38(FOE51)-0.46(UV)	0.97	0.0043
1	RUT01	EI	RUT01=4640.0+106.4(EI)	0.73	0.0295
2	RUT01	EI, UV	RUT01=107.6(ELONG)+153.2(UV)	0.85	0.0571
3	RUT01	EI, UV, FOE21	RUT01=-11448-57.9(FOE21)+81.78(EI)+397.4(UV)	0.998	0.0029
1	RUTM	UV	RUTM=0.77-0.007(UV)	0.46	0.1397
2	RUTM	UV, FOE21	RUTM=1.13+0.002(FOE21)-0.016(UV)	0.91	0.0287
1	FAT10	UV	FAT10=262972-3779.7(UV)	0.74	0.0269
2	FAT10	UV, FOE31	FAT10=372333+1406.9(FOE31)-6356(UV)	0.95	0.0117

Forward Selection Procedure

The forward selection procedure was used to develop multivariable regression equations for predicting permanent deformation or fatigue parameters from aggregate properties. The independent variables were aggregate properties as was defined in Table 10, and the dependent variables were the mix permanent deformation and fatigue properties as was defined in Table 13.

The forward selection procedure produced two- and three-variable models as described in Table 16. These models will be discussed in the following section.

Permanent Deformation. The mix validation parameters were the rut depth (GARUT) determined with the GWLT, $G^*/\sin\delta$ or stiffness of the mix at high temperature and slow loading (RUT01), and slope of the log frequency versus the log G^* regression line at high temperature (RUTM).

The results from the regression for both the GARUT and RUTM indicate that UV is the single variable most strongly related to permanent deformation (see Table 16). As UV increases (representing a more angular and/or rough-textured aggregate), both GARUT (rutting in mm after 4,000 load cycles) and RUTM (rate of development of rutting) decrease. Figure 2 compares measured GARUT values with values predicted with the one-variable model.

Flat or elongated value at the 5:1 ratio (FOE51) is an important second variable for GARUT. The coefficient of determination R^2 increases from 0.84 to 0.97 (a 15 percent increase) when FOE51 is combined with UV in the two-variable model. P also decreases from 0.0104 to 0.0043, indicating increased significance. According to this model, a high FOE51 value is not desirable. Figure 3 compares measured GARUT values with values predicted with the two-variable model.

The RUTM model, which indicates rate of development of rutting, also improves when the UV value is combined with the flat or elongated value at the 2:1 ratio (FOE21). R 2 increases from 0.46 to 0.91 (a 97 percent increase) and P improves from 0.1397 to 0.0287. According to this model, a high FOE21 value is not desirable. Figure 4 compares measured RUTM values with values predicted with the two-variable model.

The model for RUT01 shows elongation index (EI) as the primary independent variable followed by UV and flat or

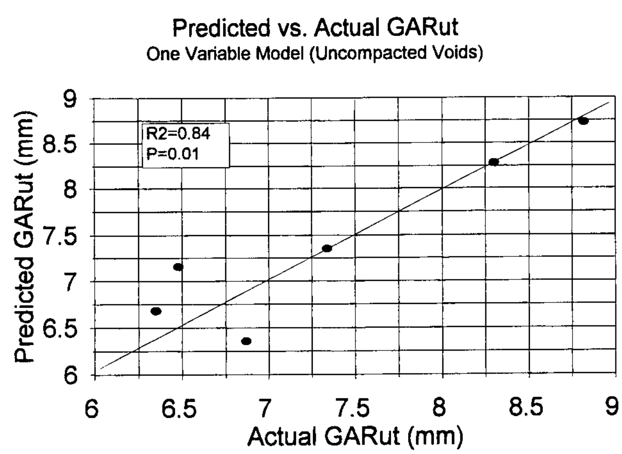


Figure 2. Predicted GARut values vs. actual GARut values using a one-variable (UV) model.

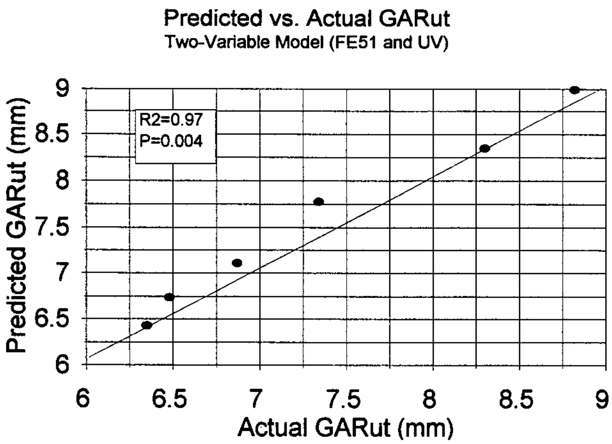


Figure 3. Predicted GARut values vs. actual GARut values using a two-variable (UV and FOE51) model.

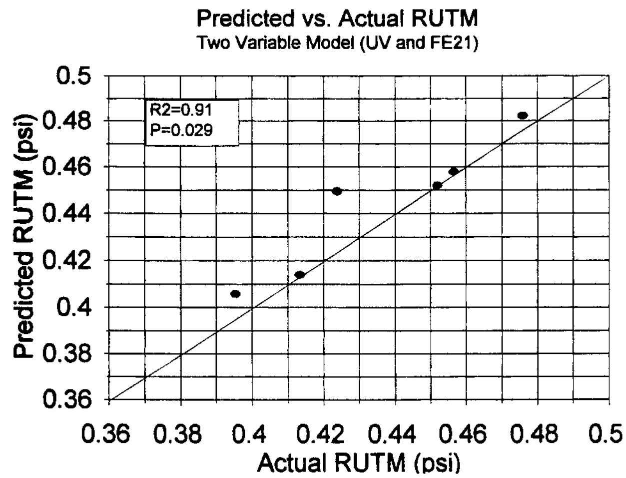


Figure 4. Predicted RUTM values vs. actual RUTM values using a two-variable (UV and FOE21) model.

elongated 2:1 (FOE21). It is not understood why EI appeared as the primary variable in this model. According to this model, higher EI results in higher RUT01 or higher resistance to rutting.

Considering the results of the three models for determining GARUT, RUTM, and RUT01 it is concluded that UV is the aggregate test which is best related to permanent deformation or rutting. FOE21 appears in two out of three rutting models. FOE21 is preferable to FOE51 because the latter has a very narrow range and does not identify the difference between aggregates as distinctly as the former. For example, of the six aggregates in the analysis, three have FOE51 of 0 percent and the remaining three have a range of 1 to 4 percent. Therefore, the recommended coarse aggregate tests which are related to permanent deformation are UV and flat or elongated particles (2:1 ratio).

Fatigue Cracking. The model for Gsinδ at 1.0 Hz (FAT10) shows UV is the aggregate test best related to fatigue cracking (Table 16). Flat or elongated 3:1 ratio (FOE31) is indicated as the secondary variable, increasing the R^2 value from 0.74 to 0.95 (a 28 percent increase) and the significance level from 0.0269 to 0.0117. Figure 5 compares measured FAT10 values with values predicted with the two-variable model.

UV and flat or elongated 3:1 are aggregate properties best related to fatigue performance. However, it is recommended that flat or elongated 2:1 (FOE21) be substituted for FOE31 because they are related (R^2=0.80, P=0.057) and FOE21 was suggested for use in determining permanent deformation. This would minimize the number of tests for characterizing coarse aggregate particle shape and surface texture.

Therefore, the recommended coarse aggregate tests which are related to fatigue cracking are UV and flat or elongated 2:1 (FOE21).

Conclusions

Table 17 shows the coarse aggregate tests that are related to performance of HMA pavements on the basis of the preceding statistical analysis. These tests are recommended for evaluating coarse aggregates for HMA in pavements.

FINDINGS OF FINE AGGREGATE PARTICLE SHAPE, ANGULARITY, AND SURFACE TEXTURE STUDY

The objective of this segment of the study was to evaluate several tests for quantifying fine aggregate particle shape, angularity, and surface texture and identify the aggregate test that is best related to permanent deformation and fatigue cracking of HMA in pavements.

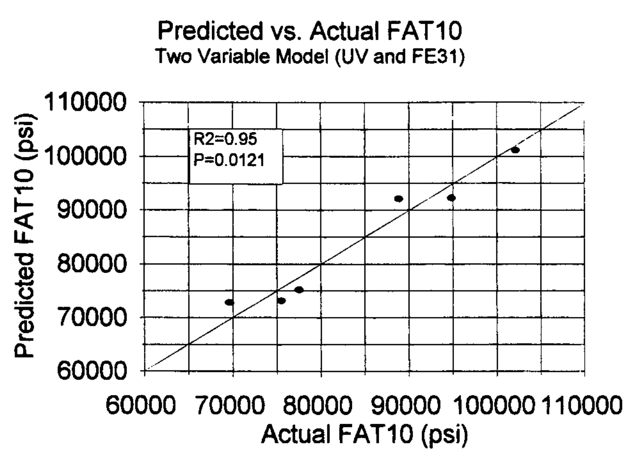


Figure 5. Predicted FAT10 values vs. actual FAT10 values using a two-variable (UV and FOE31) model.

TABLE 17 Summary of recommended coarse aggregate particle shape tests

Performance Parameter	Recommended Coarse Aggregate Test
Permanent Deformation	Uncompacted Voids (UV) Flat or Elongated Particles (2:1)
Fatigue Cracking	Uncompacted Voids (UV) Flat or Elongated Particles (2:1)

Aggregate Types and Test Results

Nine fine aggregates, described in Table 18, were chosen to provide a wide range of particle shapes, angularities, and surface textures.

The following three aggregate tests were selected to characterize the fine aggregate particle shape and texture:

Index of Aggregate Particle Shape and Texture (ASTM D 3398)
Uncompacted Voids (AASHTO T 304 or ASTM C 1252)
Particle Shape from Image Analysis (University of Arkansas Method)

None of the preceding test methods is used routinely in the United States although the UV test is now required in Superpave mix design.

Tests for specific gravity and absorption (AASHTO T 84) were conducted for mix design purposes only.

Table 19 shows the results of the fine aggregate tests. Each test was conducted in triplicate; however, only the averages are reported.

The index of aggregate particle shape and texture (ASTM D 3398) was performed on each individual size fraction of the fine aggregate (from passing 2.36 mm or No. 8 sieve to being retained on 150 µm or No. 100 sieve). The weighted average was calculated on the basis of the amount of each size fraction in the gradation of the HMA tested. This weighted average has been called the combined index of particle shape in Table 19.

UV of the fine aggregate were determined by Method A of ASTM C 1252 or AASHTO T 304.

Two shape test parameters—the EAPP index and the Roundness index—were obtained by image analysis of the fine aggregate particles as discussed earlier.

Mix Composition and Validation Tests

Figure 6 shows the gradation used for the HMA specimens. In order to maximize the amount of fine aggregate and thus increase the effect of each fine aggregate on the mix, fine aggregate was considered to be −4.75 mm (No. 4) to + 75 µm (No. 200). The selected gradation was also above the

TABLE 18 Fine aggregates description and location

Sample Number	Description	Location
FA-1	Round Natural Sand	Louisiana
FA-2	Subrounded Natural Sand	Alabama
FA-3	Subangular Natural Sand	Alabama
FA-4	Sandstone Manufactured Sand	Alabama
FA-5	Limestone Manufactured Sand	Alabama
FA-6	Dolomite Manufactured Sand	Alabama
FA-7	Granite Manufactured Sand	Georgia
FA-8	Quartzite Manufactured Sand	Georgia
FA-9	Blast Furnace Slag Manufactured Sand	Alabama

TABLE 19 Aggregate test results (fine aggregate study)

Mix #	FA-1	FA-2	FA-3	FA-4	FA-5	FA-6	FA-7	FA-8	FA-9
Test	Round Natural Sand	Sub-rounded Natural Sand	Sub-angular Natural Sand	Sand-stone Manu-factured Sand	Limestone Manu-factured Sand	Dolomite Manu-factured Sand	Granite Manu-factured Sand	Quartzite Manu-factured Sand	Blast Furnace Slag Manu-factured Sand
Combined Index of Particle Shape (#8-#100)	9.2	12.8	15.1	13.7	15.1	13.6	16.7	17.4	15.3
EAPP Index >0.75 from image Analysis	0.695	0.289	0.230	0.092	0.235	0.171	0.122	0.172	0.090
Roundness Index >0.7 from Image Analysis	0.447	0.373	0.367	0.306	0.176	0.314	0.229	0.217	0.287
Uncom-pacted Voids	40.3	43.3	46.1	44.8	46.0	46.0	47.3	47.5	46.1
Apparent. SG	2.658	2.652	2.675	2.646	2.744	2.869	2.750	2.711	2.794
Bulk SG	2.626	2.625	2.649	2.488	2.665	2.786	2.657	2.623	2.527
Water Absorption	0.5	0.4	0.4	2.4	1.1	1.1	1.3	1.2	3.8

Superpave-restricted zone to maximize the effect of fine aggregate in the dense-graded HMA. The material passing 12.5 mm (½ in.) and retained on 4.75 mm (No. 4) was held constant for each mix. The coarse aggregate was a round uncrushed gravel. This source was chosen in order to maximize the effect of the fine aggregate in the mixture validation tests. A Superpave PG 64-22 grade-asphalt cement was used for all mix testing. The Pine gyratory compactor was used to compact specimens for Superpave volumetric mix design. HMA mixtures were aged for 4 hr at 135°C (275°F) before compaction. Two specimens (150-mm diameter by 115-mm height) were compacted at each of four asphalt contents. Asphalt contents which gave 4 percent air voids at N_{design} (119 gyrations, for intermediate design traffic level of 10^7 $EASL_s$) were selected as the optimum asphalt contents. Specimens with optimum asphalt content were tested by the SST (simple shear at constant height and frequency sweep at constant height) and the IDT. The effective temperature for permanent deformation (T_{eff} [PD]) was 40°C. The effective temperature for fatigue cracking (T_{eff} [FC]) was 20°C. These samples were also aged for 4 hr, then heated to compaction temperature and compacted. The testing of these specimens was performed by the Asphalt Institute.

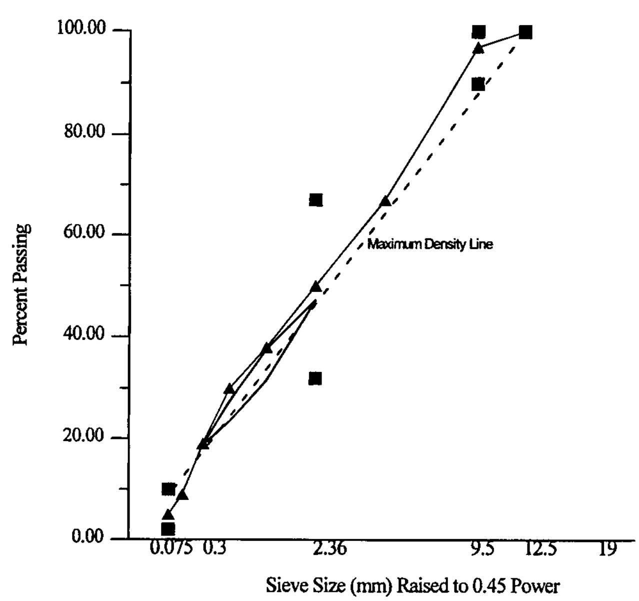


Figure 6. Gradation of HMA mixtures (fine aggregate, plastic fines, and P200 studies).

TABLE 20 Mix data for SST tests (fine aggregate study)

Mix	Optimum Asphalt Content	Theoretical Maximum Density	Average VTM	Average VMA
FA-1	3.6	2.523	7.3	14.3
FA-2	4.5	2.484	7.0	16.5
FA-3	4.7	2.487	7.0	17.0
FA-4	5.6	2.442	6.7	16.0
FA-5	5.0	2.511	6.3	16.1
FA-6	4.3	2.616	6.6	14.4
FA-7	5.2	2.499	6.8	17.0
FA-8	5.6	2.462	4.8	16.2
FA-9	7.0	2.455	5.5	16.4

Table 20 shows the optimum percent asphalt cement, theoretical maximum density (TMD), average VTM, and the average VMA for all nine mixture specimens tested.

Specimens were also made to be tested in the GLWT. Cylindrical specimens for this test were compacted by Pine gyratory compactor and tested at the Georgia DOT Asphalt Laboratory. Table 21 contains the average VTM and VMA data for the specimens. Each cylindrical specimen was subjected to 8,000 cycles at 40°C, 690 kPa (100 psi) pressure in hoses, and a wheel load of 45 kg (100 lb). The rut depth (GARUT) was measured manually at 1,000-cycle intervals. A test temperature of 40°C was necessary because testing done at 50°C did not give consistent results although attempted twice.

Superpave intermediate mix evaluation was used to determine the propensity of the HMA mixtures for permanent deformation (rutting) and fatigue cracking. The following three test parameters as discussed in detail under coarse aggregate particle shape and texture were used:

1. *G*/sinδ* at 0.1 Hz (RUT01)
2. *Slope (m)* of the frequency versus *G* plot* (RUTM)
3. *G*sinδ* at 1.0 Hz (FAT10)

Mix validation parameters from the SST and GLWT are summarized in Table 22.

Statistical Analysis

A correlation matrix was developed for all aggregate and mix validation tests using the SAS program. This was done to find out which aggregate characterization test methods give comparable results and correlate best with mix validation properties. Table 23 shows the correlation matrix for aggregate and mix properties. The correlation coefficient (R value) is the top number in each cell, and the bottom number is the statistical significant level (P) corresponding to the correlation coefficient.

Only seven of the nine fine aggregates were included in the correlation matrix shown in Table 23. These same seven aggregates were also the only ones included in all subsequent analysis. FA-2 and FA-6 were excluded from the analysis for the reasons discussed below.

FA-2 (Subrounded Natural Sand)

The specimens made with FA-2 for the GLWT had the highest air voids (7.6 percent) compared with the other fine aggregate mixes (Table 21) and the highest rut depth apparently because of additional consolidation, which is not representative of the mix's rutting potential. As discussed later, the values obtained from the GLWT are the primary mix validation results used in this element of the study and, therefore, FA-2 was deleted from the statistical analysis.

FA-6 (Dolomite Manufactured Sand)

The specimens made with FA-6 used in the GLWT had the least rut depth. All four particle shape test parameters indicate its rutting potential should be moderate. It

TABLE 21 Mix data for GLWT cylindrical specimens (fine aggregate study)

Mix	Average VTM	Average VMA
FA-1	5.8	12.9
FA-2	7.6	17.0
FA-3	6.8	16.8
FA-4	5.7	15.0
FA-5	5.8	15.6
FA-6	7.0	14.8
FA-7	5.5	15.9
FA-8	6.1	17.4
FA-9	6.2	17.1

TABLE 22 Fine aggregate mixture validation test results

Mix	Rutting Parameters			Fatigue Parameter
	High Temperature (40°C)			Intermediate Temperature (20°C)
	GLWT Rut Depth (mm)	G*/sinδ @ 0.1 hz psi	m	G*sinδ @ 1.0 hz psi
FA-1	1.94	9200	0.43975	84428
FA-2	4.04	7757	0.40405	93327
FA-3	1.76	7095	0.41035	80871
FA-4	1.59	9391	0.41720	76661
FA-5	1.57	9345	0.41325	86735
FA-6	0.43	11339	0.45740	111609
FA-7	0.75	9134	0.43575	79765
FA-8	0.86	14011	0.43730	94191
FA-9	1.52	9913	0.41360	79488

appears the mix containing FA-6 was very stiff because its asphalt content of 4.3 percent is too low for an angular fine aggregate. This aggregate (dolomitic limestone) was also excluded in the coarse aggregate element of this study.

Correlation Between Aggregate Tests

Table 23 shows that UV has an excellent correlation with index of particle shape and surface texture (Index) (R=0.994, P=0.0001) as also shown in Figure 7. UV also has a good correlation with the EAPP index (R = −0.874, P=0.01) and the Roundness index (Round) (R=−0.797, P=0.03). UV and Index measure the combined effect of particle shape and surface texture, whereas the EAPP index and the Roundness index (being obtained by imaging techniques) primarily measure particle shape. Index also has a good correlation with the EAPP index (R=−0.84, P=0.02) and the Roundness index (R=−0.805, P=0.03).

Correlation Between Aggregate Tests and Mix Properties

Table 23 shows that the correlations between all four particle shape test parameters and the SST parameters are poor. This is unlike the coarse aggregate study where reasonably

TABLE 23 Correlation matrix betwene fine aggregate properties and mix properties

	Index	UV	EAPP	Round	GARut	Rut01	Rutm	Fat10
Index	1.0	0.994 0.0001	-0.840 0.02	-0.805 0.03	-0.816 0.03	0.380 0.40	-0.172 0.71	0.226 0.63
UV		1.0	-0.874 0.01	-0.797 0.03	-0.776 0.04	0.288 0.53	-0.260 0.57	0.139 0.77
EAPP			1.0	0.682 0.09	0.621 0.14	-0.155 0.74	0.460 0.30	0.232 0.62
Round				1.0	0.746 0.05	-0.445 0.32	0.120 0.80	-0.324 0.48
GARut					1.0	-0.556 0.20	-0.355 0.43	-0.256 0.58
Rut01						1.0	0.502 0.25	0.741 0.06
Rutm							1.0	0.438 0.33
Fat10								1.0

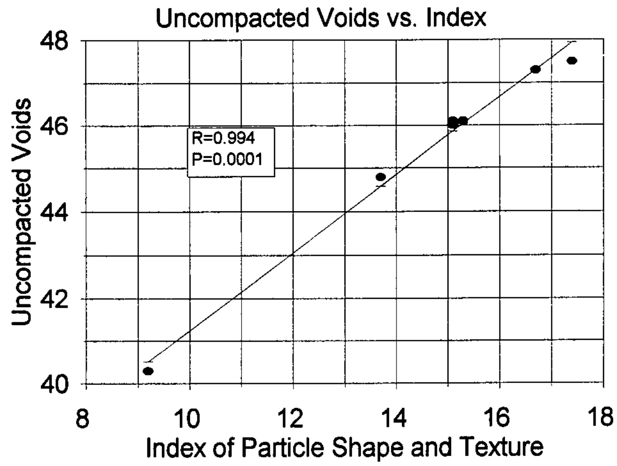


Figure 7. Uncompacted voids vs. index of particle shape and texture (fine aggregate study).

good correlations were obtained. This difference is thought to result from differences in the level of compaction used for preparing specimens. The average air voids in the SST specimens used in the coarse aggregate study and the fine aggregate study were 3.8 percent and 6.4 percent, respectively. Superpave intermediate mix analysis requires air voids of compacted SST specimens to be close to 7 percent. However, it appears that the influence of aggregate particle shape and surface texture is accentuated or mobilized when the aggregate particles are packed closer together (that is, 4 percent rather than 7 percent air voids). This is the likely explanation for the good correlation obtained with SST results in the case of the coarse aggregate study where the samples were compacted to 3.8 percent average air voids. This phenomenon was also observed by other researchers (*53*). In view of the poor correlations, SST parameters will not be used as mix validation parameters in the fine aggregate study. Only the GARUT will be used to relate fine aggregate particle shape tests to permanent deformation or rutting.

Correlations, in decreasing order, of the particle shape tests or parameters with the GARUT are shown in Table 24.

Index has the best correlation with rutting (R=−0.816, P=0.025), closely followed by UV (R=−0.776, P=0.040). Correlation with the Roundness index and the EAPP index are not as strong with numerically lower coefficients of correlation and significant levels less than 5 percent.

TABLE 24 Correlation between aggregate parameters and rut depth (fine aggregate study)

Parameter	Correlation Coefficient R	Significance Level P
Index	-0.816	0.025
UV	-0.776	0.040
Roundness Index	0.746	0.054
EAPP	0.621	0.136

Forward Selection Multiple Regression Procedure

In this procedure, the dependent variable was the mix rut depth in mm obtained by the GLWT and four independent variable aggregate tests: Index, UV, EAPP, and Roundness. No variable was found that would significantly improve the following one-variable equation:

$$GARUT = 2.993 - 0.102\ (\text{Index})$$

$$R^2 = 0.665$$

$$P = 0.025$$

Index is the fine aggregate parameter which is best related to performance of HMA in terms of permanent deformation. However, UV is recommended rather than Index for the following reasons:

- UV has an excellent correlation with Index (R=0.994, P=0.0001)(see Figure 7).
- UV has the second best correlation with the GLWT rut depth (R=−0.776), which is only slightly less than R = −0.816 for Index and which is significant at the 4 percent level (see Figure 8).
- UV is more practical than Index because it is significantly less time consuming. Testing replicates of a fine aggregate sample for UV takes only 1 hr whereas testing for Index takes about 8 hours (bulk specific gravity must be known for either test).

Conclusions

UV is recommended as a test for quantifying fine aggregate particle shape, angularity, and surface texture. UV was se-

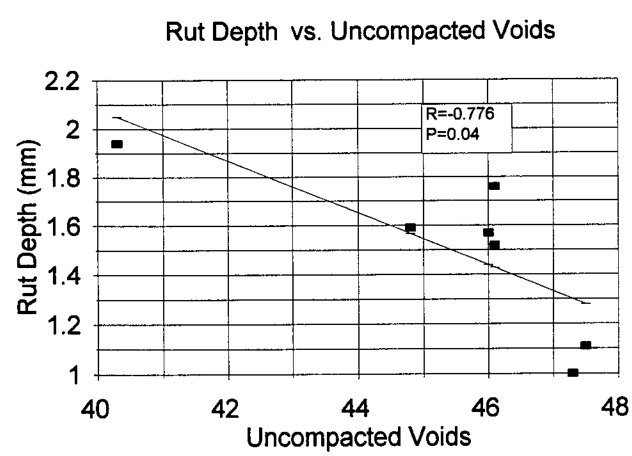


Figure 8. Uncompacted voids vs. rut depth (fine aggregate study).

TABLE 25 Aggregate types and location (plastic fines study)

Sample Number	Description	Location
PF-1	Pit Run Natural Sand	Alabama
PF-2	High Calcium Limestone	Alabama
PF-3	Dolomite	Alabama
PF-4	Granite	Georgia
PF-5	Blast Furnace Slag	Alabama
PF-6	Limerock	Florida
PF-7	Granite with 4% High Plasticity Clay	Georgia
PF-8	Limerock with 3.4% High Plasticity Clay	Florida
PF-9	Granite with 2.6% High Plasticity Clay	Georgia
PF-10	Limerock with 1% High Plasticity Clay	Florida

lected on the basis of permanent deformation without any consideration of fatigue cracking. The UV test is recommended for evaluating fine aggregates for HMA in pavements.

FINDINGS OF STUDY ON PLASTIC FINES IN THE FINE AGGREGATE

The presence of plastic fines in the fine aggregate portion of HMA may induce stripping in the mix when exposed to water or moisture. This section focuses on determining the aggregate test method or methods that best indicate the presence of detrimental plastic fines (which, in turn, induce stripping) in HMA mixtures. Stripping may lead to permanent deformation in HMA pavement.

Aggregate Types and Test Results

Six fine aggregates were chosen to give a wide range of mineralogical compositions and sand equivalent values. In addition, four fine aggregate blends were made to produce a wider range of sand equivalent values. These fine aggregates were obtained by blending a high plasticity clay with fine aggregates PF-4 (Granite) and PF-6 (Limerock). Table 25 identifies the ten fine aggregates.

Table 26 summarizes measured properties. Each test was run in triplicate and average values are reported. Specific gravity and water absorption of fine aggregate (AASHTO T 84) were determined for mix design purposes only.

As shown in Table 26, the plasticity index for all of these aggregates, especially the fraction passing No. 40 sieve, was

TABLE 26 Aggregate test results (plastic fines study)

Mix #	PF-1	PF-2	PF-3	PF-4	PF-5	PF-6	PF-7	PF-8	PF-9	PF-10
Test	Natural Sand	Limestone	Dolomite	Granite	Blast Furnace Slag	Limerock	Granite +4% Clay	Limerock +3.4% Clay	Granite +2.6% Clay	Limerock +1% Clay
Sand Equivalent	24	90	91	58	87	84	39	63	51	74
Plasticity Index (Passing #40)	NP	NP	NP	NP	NP	NP	NP	NP	NP	NP
Plasticity Index (Passing #200)	29	NP	NP	NP	NP	NP	40	40	40	40
Methylene Blue	18.4	1.3	0.3	2.1	2	9.5	80.0	66.0	47.5	26.9
Apparent SG.	2.667	2.744	2.869	2.750	2.711	2.678	2.730	2.672	2.823	2.676
Bulk SG.	2.558	2.665	2.786	2.657	2.623	2.358	2.651	2.363	2.743	2.360
Water Absorption	2.0	1.1	1.1	1.3	1.2	5.1	-	-	-	-

TABLE 27 Mix design data (plastic fines study)

Mix Designation	Optimum Asphalt Content	VTM	VMA
PF-1	5.5	4.0	14.2
PF-2	5.3	4.0	15.8
PF-3	3.9	4.0	11.9
PF-4	5.2	4.0	14.4
PF-5	6.8	4.0	15.2
PF-6	8.0	4.0	20.6
PF-7	5.2	4.0	14.4
PF-8	8.0	4.0	20.6
PF-9	5.2	4.0	14.4
PF-10	8.0	4.0	20.6

zero (nonplastic); therefore, the two main aggregate tests that were used for comparative purposes in this section were the sand equivalent test and the methylene blue test. Sand equivalence measures the relative amount of clay-sized particles in a fine aggregate. The methylene blue test determines the amount and nature of potentially detrimental material, such as clay and organic material, that may be present in an aggregate.

Mixture Tests and Results

Superpave volumetric mix design was used to determine the optimum asphalt content to ensure that each fine aggregate mix contains 4 percent air voids. The gradation was held constant for all mixes (see Figure 6). All mixes contained a common limestone coarse aggregate (33 percent). The mix design data, such as optimum asphalt content, VTM, and VMA, are given in Table 27.

Two mixture validation tests were used in this section to evaluate the stripping potential. The Hamburg wheel tracking test is conducted under water maintained at 50°C and can indicate a mix's resistance to stripping. The inflection point is the parameter of interest in this test. AASHTO T 283 was also used as a mixture validation test to determine the moisture susceptibility of the HMA mixtures in terms of tensile strength ratio (TSR). Table 28 contains the average specimen VTM and VMA information for the slabs used in the Hamburg wheel tracking tests.

Table 29 contains the average specimen VTM and VMA information for the specimens used in the AASHTO T 283 tests.

TABLE 28 VTM and VMA for Hamburg wheel tracking specimens

Mix Designation	Optimum Asphalt Content	VTM	VMA
PF-1	5.5	6.0	16.0
PF-2	5.3	5.0	15.7
PF-3	3.9	7.1	14.9
PF-4	5.2	4.6	15.0
PF-5	6.8	5.4	15.6
PF-6	8.0	5.0	21.6
PF-7	5.2	5.0	15.5
PF-8	8.0	5.8	22.2
PF-9	5.2	4.2	14.9
PF-10	8.0	5.0	21.5

TABLE 29 VTM and VMA for AASHTO T 283 specimens

Mix Designation	Conditioned Specimens		Unconditioned Specimens	
	VTM	VMA	VTM	VMA
PF-1	6.9	16.8	6.9	16.8
PF-2	7.2	17.7	7.4	17.9
PF-3	7.0	14.7	7.0	14.8
PF-4	7.0	17.1	7.0	17.1
PF-5	7.3	17.3	7.2	17.2
PF-6	7.4	23.6	7.3	23.5
PF-7	7.4	17.6	7.3	17.6
PF-8	7.1	23.3	7.1	23.2
PF-9	8.0	18.3	8.4	18.6
PF-10	6.5	23.0	6.5	22.8

Mixes PF-6, PF-8, and PF-10 used Florida limerock as the main fine aggregate. Mixes PF-8 and PF-10 only differ from PF-6 in that they contain a small amount of clay. These three mixes have very high VMA and high optimum asphalt content (Table 27). The only criteria used to determine the optimum asphalt content of these mixes was 4 percent VTM. It is very likely that this limerock degraded excessively in the gyratory compactor. The resulting degradation will increase the VMA of the mix, thus requiring an increased asphalt content in order to achieve the proper VTM. Moreover, the limerock is highly absorptive and, therefore, required additional asphalt binder.

Table 30 contains the mixture validation test results.

TABLE 30 Mixture validation test results (plastic fines study)

Mix Designation	Asphalt Content at 4% Voids	TSR Results, %	Hamburg Wheel Tester (Inflection Point)
PF-1	5.5	49.3	5000
PF-2	5.3	85.0	17000
PF-3	3.9	79.1	17500
PF-4	5.2	61.3	6800
PF-5	6.8	47.5	17000
PF-6	8.0	62.0	3800
PF-7	5.2	44.2	5500
PF-8	8.0	48.4	5200
PF-9	5.2	30.8	4300
PF-10	8.0	49.0	2400

Statistical Analysis

Table 31 contains a correlation between the aggregate tests and mix validation tests. In addition to the comparison with sand equivalent and methylene blue, the mix validation tests were also compared with variations of the results of these two aggregate tests, in the event that the relationships may not be linear. The variations used in the comparison were the log of each test result, the square of each result, and the inverse of the square of the results.

The TSR is best related to Log methylene blue ($R=-0.79$, $P=0.006$). Its correlation with the square of sand equivalent is $R=0.67$, $P=0.03$. The inflection point is also best related to Log methylene blue ($R=-0.82$, $P=0.003$). Its correlation with the square of sand equivalent is $R=0.69$, $P=0.03$.

Forward Selection Multiple Regression Procedure

The forward selection procedure was used to determine if combinations of variables containing methylene blue and sand equivalent values could improve prediction of the TSR and inflection point. No combination was found that improved the predictability of the relationships with Log methylene blue shown in Table 32. Figures 9 and 10 show the plots of Log methylene blue versus TSR and inflection point, respectively.

Conclusions

Both TSR and inflection point test data indicate that methylene blue is the fine aggregate test which is best related to stripping of HMA. Therefore, the methylene blue test is the only test selected to indicate the presence of detrimental plastic fines which may induce stripping in HMA mixtures, which, in turn, may lead to permanent deformation of HMA

TABLE 31 Correlation between aggregate properties and mix properties (plastic fines study)[a]

Aggregate Test	TSR	Inflection Point
Sand Equivalent	0.615 0.06	0.618 0.06
Methylene Blue	-0.639 0.05	-0.552 0.098
(Sand Equivalent)^2	0.673 0.03	0.686 0.03
(Methylene Blue)^2	-0.502 0.14	-0.390 0.27
Log(Sand Equivalent)	0.525 0.12	0.528 0.12
Log(Methylene Blue)	-0.794 0.006	-0.825 0.003
1/(Sand Equivalent)^2	-0.316 0.37	-0.340 0.34
1/(Methylene Blue)^2	0.538 0.11	0.559 0.09

[a]Top values are correlation coefficients R and bottom values are significance levels P in each cell.

pavements. The methylene blue test is recommended for evaluating fine aggregates for HMA in pavements.

FINDINGS OF TOUGHNESS AND ABRASION RESISTANCE AND DURABILITY AND SOUNDNESS STUDY

Sixteen common aggregates were selected for both toughness and abrasion resistance and durability and soundness studies. These aggregates have a range of historical performance from good to poor as judged by state DOT materials engineers. Aggregate sources are located in all four climatic regions as delineated in the SHRP studies with 10 of 16 aggregates from the wet-freeze region. Aggregates used in controlled field pavement performance studies (Minnesota test road and Westrack in Nevada) were included. The selected aggregates have a wide range of mineralogical compositions (rock types). However, the volume of use was also factored into the selection process. Consequently, five sources of crushed carbonate stone and two sources of gravel with some limestone were selected.

The 16 selected sources of aggregates include five carbonate sources, four gravels, two granites, one traprock, one siltstone, one sandstone, one basalt, and one steel slag. Table 33 presents general aggregate descriptions.

Toughness and Abrasion Resistance Data

The following five test methods were used to determine the toughness and abrasion resistance of the 16 aggregates:

- Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine (AASHTO T 96)
- Aggregate Impact Value (British Standard 812: Part 3)
- Aggregate Crushing Value (British Standard 812: Part 3)

TABLE 32 Results of forward selection procedure (plastic fines study)

Step	Dependent Variable	Independent Variable	Model	R^2	P
1	TSR	Log Methylene Blue	TSR = 70.277-6.84(Log Methylene Blue)	0.63	0.006
1	Inflection Point	Log Methylene Blue	Inflection Point=14104.2-2644.4(Log Methylene Blue)	0.68	0.003

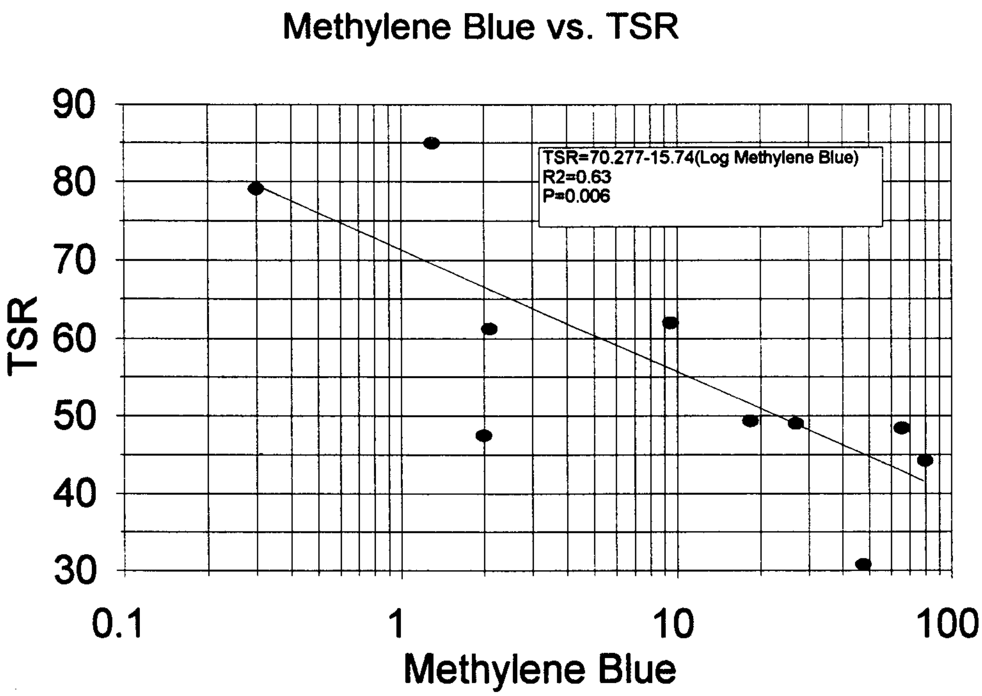


Figure 9. Methylene Blue vs. TSR (plastic fines study).

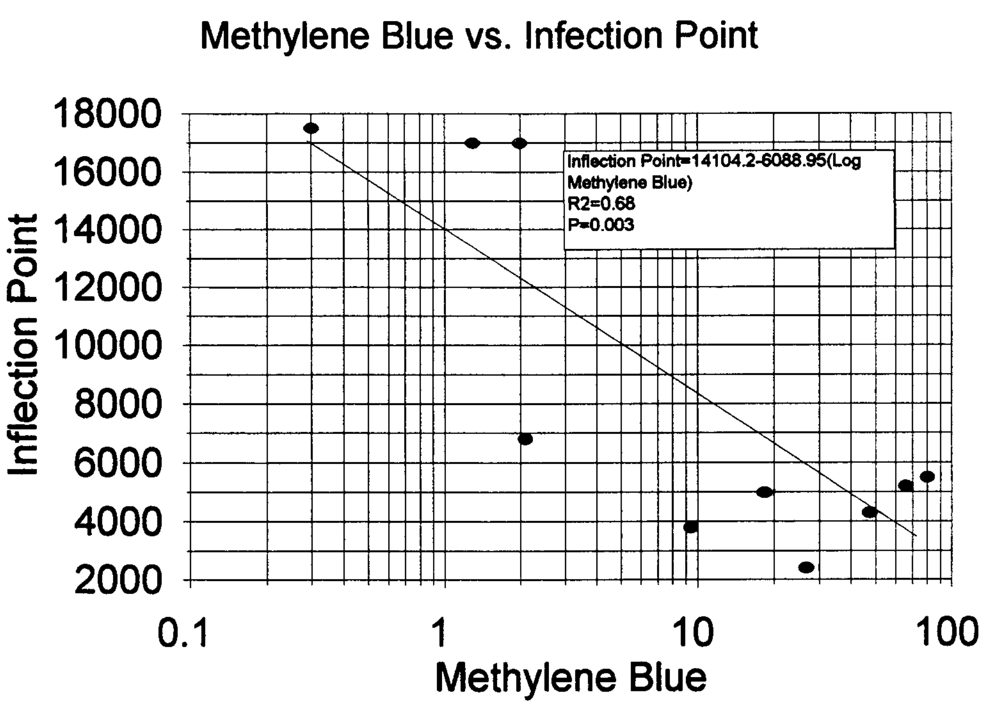


Figure 10. Methylene Blue vs. Inflection point (plastic fines study).

TABLE 33 General aggregate description (toughness and soundness study)

Rock Type and State	Description
1. Traprock, NY	A fine-grained gabbroic rock with plagiclase phenocrysts in a diabasic texture, forming an interlocking mesh of randomly-oriented crystals that surround the smaller Pyroxene.
2. Granite, GA	Consists of coarsely grained granitic genesis, with the gneissosity defined by layers rich in biotite, and occasionally hornblende, that alternate with layers rich in quartz and feldspars.
3. Steel Slag, IN	Comprised of cm-scale pieces of steel slag. Most pieces are grey, round, and composed of finely-grained silicates and iron-titanium oxide minerals. The material fractures easily with a mortar and pestle into sand-sized grains.
4. Gravel, MN	A gravel from a glacial terrain, and containing pieces of differing igneous, metamorphic, and sedimentary rocks; syenite, gabbro, feld-spathic gneiss, glauconitic sandstone, and fossiliferous limestone.
5. Gravel, NV	Comprised of grey to brown hornblende-pyroxene andesites. The mineralogy of the aggregate is uniform relative to other gravels, although it has considerable variation in modal percentages, grain size, and color.
6. Limestone, IA	A dolostone composed entirely of fine to medium-grained (0.1 to 0.3 mm wide) dolomite crystals.
7. Granite, SC	A gneiss with cm- scale compositional layering defined by variations in biotite and feldspars, which is most evident in the largest pieces of the aggregate. Modal variations in biotite and feldspars results in aggregate pieces of feldspathic- and biotite-gneiss.
8. Gravel, MN	A gravel with approximately 70% limestone and claystone, and 30% felsic gneiss, gabbro, and granite.
9. Limestone, IA	Contains pieces of both fossiliferous limestone and finely-grained dolostone.
10. Gravel, PA	A fine to medium grained gravel and containing detrital quartz, mica, metamorphic rock fragments, and feldspars that are cemented by quartz and hematite.
11. Limerock, FL	Comprised of two types sedimentary rocks: about 90% of the aggregate is limestone, and approximately 10 % is chert composed of precipitated quartz.
12. Limestone, TX	A fossiliferous limestone, with detrital clasts including shell fragments and glauconite pellets, cemented by coarse calcite.
13. Sandstone, PA	A fine to medium grained sandstone containing detrital quartz, plagioclase, microcline, biotite and muscovite. Some pieces also contain plant fossils and amorphous carbon.
14. Limestone, MN	A micritic limestone with varying proportions of fossil fragments, and fossiliferous limestone with secondary dolomite.
15. Siltstone, VA	A hard and fissle, purple-weathering slate. The slate has a uniform size that is mostly in the clay-range. Pieces of the slate fracture readily along the foliation into rectangular shapes.
16. Basalt, OR	A finely grained basalt in which the primary mineralogy has been extensively altered to a mixture of micas. The micas, including mixed clays and chrolite surround most grains in the rock and limit its strength.

- Superpave Gyratory Compactor
- Micro-Deval Abrasion Test

Of the preceding five test methods, only one (the Los Angeles abrasion test) is commonly used in the United States. Tables 34 through 37 summarize results from toughness and abrasion resistance tests. The values in these tables are the average of three replications. AASHTO 8 gradation was used in all tests. Large size materials (AASHTO 57) were tested for 4 of the 16 total sources. A brief discussion of the test data follows.

Degradation in Superpave Gyratory Compactor

Data from the Superpave gyratory compactor is illustrated in Figure 11. The gradation curve for the aggregate before compaction (solid line) is for one sample, whereas, the curve

TABLE 34 Superpave gyratory compactor bare aggregate degradation test results

Aggregate Sources	Degradation of Bare Aggregate in Superpave Gyratory Compactor, Δ% Retained on 4.75 mm (#4) Sieve*			
	AASHTO No.57	AASHTO No.8	AASHTO No.57 + Fine Aggregate	AASHTO No.8 + Fine Aggregate
1. Traprock, NY	**	12.5	**	2.7
2. Granite, GA	9.9	15.4	4.2	3.6
3. Steel Slag, IN	**	7.9	**	1.4
4. Gravel, MN	**	13.9	**	2.4
5. Gravel, NV	**	10.4	**	1.3
6. Limestone, IA	14.9	16.5	5.9	3.5
7. Granite, SC	22.0	24.0	8.2	5.8
8. Gravel, MN	**	13.8	**	2.0
9. Limestone, IA	**	16.5	**	4.1
10. Gravel, PA	**	12.9	**	2.3
11. Limerock, FL	19.4	24.4	8.1	6.2
12. Limestone, TX	**	18.0	**	2.8
13. Sandstone, PA	**	13.8	**	2.1
14. Limestone, MN	**	24.6	**	5.5
15. Siltstone, VA	**	9.1	**	2.0
16. Basalt, OR	**	28.7	**	7.6

*Δ% Retained on 4.75 mm (#4) sieve is calculated as change in percent retained on 4.75 mm (#4) sieve, obtained from gradation curves, before and after compaction, and, therefore, a measure of degradation.
**Large size materials (AASHTO 57) tested for only 4 of 16 sources.

for aggregate or mix after compaction (dashed line) is the average for three samples. The gradation curves were used to quantify degradation and produce the measures in Tables 34 and 35.

Two methods were tried for the measurement of degradation. One is the sum of the gradation change and the other is the gradation change in a single representative sieve, such as 4.75 mm (No. 4) sieve. The sum of the gradation change represents the area between the compacted and uncompacted gradation curves. The difference between before and after compaction percent retained (percent passing is equivalent) on the 19.0 mm (¾ in.), 12.5 mm (½ in.), 9.5 mm (⅜ in.), 4.75 mm (No. 4), 2.36 mm (No. 8), 1.18 (No. 16), 600 μm (No. 30), 300 μm (No. 50), 150 μm (No. 100), and 75 μm (No. 200) sieves were computed and summed. The process for the 12.5 mm (½ in.) and 4.75 mm (No. 4) sieves are illustrated in Figure 11. Values for the AASHTO No. 57 size material will be larger than the AASHTO No. 8 size material because more sieve sizes are used. Therefore, the cumulative percentages obtained in this manner indicate only the relative amount of degradation for specific size material. The values for the two sizes tested are not comparable.

For this reason, another method which will provide a measure of the degradation using a single representative sieve was investigated. The effects of aggregate top size (using bare aggregate) were assessed and evaluated for the sum of changes in all sieves, the change in 2.36 mm (No. 8) sieve, and the change in 4.75 mm (No. 4) sieve.

Figure 12 shows a decrease in the degradation measured with the change in 4.75 mm (No. 4) sieve as the aggregate top size increases for open-graded gradation (no fine aggregate) (refer to Figure 12[a]); but an opposite trend for dense-graded gradation (with fine aggregate) (refer to Figure 12[b]), which indicates the degradation measured with the change in 4.75 mm (No. 4) sieve was the least aggregate top size sensitive.

In addition to the least size sensitivity in the degradation measured with the change in 4.75 mm (No. 4) sieve, the slope of virgin gradation curve for open-graded materials of both AASHTO No. 57 and AASHTO No. 8 changes dramatically at 4.75 mm (No. 4) sieve. All these merits made the change

TABLE 35 Superpave gyratory compactor mix degradation test results

Aggregate Sources	Degradation of Asphalt Concrete Mix in Superpave Gyratory Compactor, Δ% Retained on 4.75 mm (#4) Sieve*			
	AASHTO No.57	AASHTO No.8	AASHTO No.57 + Fine Aggregate	AASHTO No.8 + Fine Aggregate
1. Traprock, NY	**	8.5	**	2.2
2. Granite, GA	6.2	10.5	3.2	5.7
3. Steel Slag, IN	**	5.0	**	1.3
4. Gravel, MN	**	6.4	**	2.2
5. Gravel, NV	**	6.1	**	2.0
6. Limestone, IA	13.7	15.4	5.3	2.8
7. Granite, SC	17.9	19.2	6.7	4.1
8. Gravel, MN	**	7.6	**	1.4
9. Limestone, IA	**	16.7	**	4.8
10. Gravel, PA	**	9.1	**	2.0
11. Limerock, FL	16.8	25.4	6.9	6.3
12. Limestone, TX	**	11.6	**	2.3
13. Sandstone, PA	**	10.8	**	2.3
14. Limestone, MN	**	16.0	**	0.9
15. Siltstone, VA	**	3.6	**	1.6
16. Basalt, OR	**	26.0	**	6.9

*Δ% Retained on 4.75 mm (#4) sieve is calculated as change in percent retained on 4.75 mm (#4) sieve, obtained from gradation curves, before and after compaction, and therefore, a measure of degradation.
**Large size materials (AASHTO 57) tested for only 4 of 16 sources.

in 4.75 mm (No. 4) sieve a logical choice as an indicator of degradation.

The correlation between the degradation measured with the gradation change in the 4.75 mm (No. 4) sieve and the degradation measured with the sum of the gradation change in all sieves is shown in Figure 13 for open-graded bare aggregate (AASHTO No. 8). The R value of 0.927 (R^2= 0.859) for open-graded aggregate indicates that a good correlation exists between the gradation change in the 4.75 mm (No. 4) sieve and the sum of gradation changes in all sieves. This good correlation obtained with the 4.75 mm (No. 4) sieve and its insensitivity to aggregate top size, make it the best choice to measure the aggregate degradation during compaction in the Superpave gyratory compactor.

In summary, the test results indicate the following:

- The Superpave gyratory compactor can distinguish tough and weak aggregates. Gradation changes before and after compaction are generally greater for weaker, less abrasion-resistant aggregates than for tougher aggregates. This will be verified later when detailed correlations are developed with other toughness and abrasion tests results, such as the Los Angeles abrasion loss and aggregate impact values.
- Open-graded aggregate degrades more in the Superpave gyratory compactor than do dense-graded aggregates. The number of contact points is more numerous for dense-graded aggregates because of the presence of fine aggregate and any applied force will be distributed to many more points at much less intensity than for more open-graded aggregate.
- Incorporation of asphalt cement with aggregate reduced degradation more for open gradations than for dense gradations.

The test results for Micro-Deval, Los Angeles abrasion, aggregate impact value, and aggregate crushing value are presented in Tables 36 and 37. The British tests, aggregate impact value and aggregate crushing value, give similar test values regardless of aggregate type. Both have good repeatability. Equipment for the impact test is simple; the test is fast and does not require a laboratory compression machine.

There was good repeatability for the Micro-Deval test. However, after some testing with the device, relative slip was

TABLE 36 Micro-Deval and Los Angeles abrasion test results

Aggregate Sources	Micro-Deval (% Loss) (Canadian)		Los Angeles Abrasion (% Loss) AASHTO T 96	
	19.0 - 9.5 mm	13.2 - 4.75 mm	Procedure B (19 - 9.5 mm)	Procedure C (9.5 - 4.75 mm)
1. Traprock, NY	**	7.1	**	17.4
2. Granite, GA	4.9	6.5	19.3	24.5
3. Steel Slag, IN	**	9.9	**	14.5
4. Gravel, MN	**	9.6	**	21.2
5. Gravel, NV	**	15.2	**	19.5
6. Limestone, IA	14.6	15.0	32.7	30.0
7. Granite, SC	13.2	15.6	53.7	49.0
8. Gravel, MN	**	14.6	**	25.0
9. Limestone, IA	**	13.0	**	26.4
10. Gravel, PA	**	33.4	**	25.5
11. Limerock, FL	16.5	22.0	40.2	41.6
12. Limestone, TX	**	18.1	**	27.4
13. Sandstone, PA	**	34.0	**	27.6
14. Limestone, MN	**	28.9	**	28.3
15. Siltstone, VA	**	8.0	**	12.9
16. Basalt, OR	**	35.2	**	31.9

** Large size materials (AASHTO 57) tested for only 4 of 16 sources.

observed between the stainless steel jars and the rotating rubber roller drive. This slip caused some variability as the specified rotations were not reached. Periodic roughening of the surfaces of the rubber rollers corrected this problem. For this reason, modifications should be made to the Micro-Deval machine to prevent slip or to count the jar revolutions.

The abrasion loss for large aggregate (AASHTO No. 57) in the Micro-Deval test was consistently smaller than the loss for smaller aggregate (AASHTO No. 8). This was not the case for the Los Angeles abrasion test and is thought to result from lack of consistency in ball charge and particle surface area. In the Los Angeles abrasion test the number of balls is adjusted from 11 for procedure B (aggregate top size of ¾ in.) to 8 for procedure C (aggregate top size of ⅜ in.). However, in the Micro-Deval test, the ball charge is the same (5,000 gm) for both aggregate sizes (19.0- and 13.2-mm top sizes).

Durability and Soundness Data

The following test methods were used to determine the durability and soundness of the 16 aggregates:

- Soundness of Aggregate by Use of Sodium Sulfate (AASHTO T 104)
- Soundness of Aggregate by Use of Magnesium Sulfate (AASHTO T 104)
- Soundness of Aggregate by Freezing and Thawing (AASHTO T 103 - Procedure A)
- Soundness of Aggregate by Freezing and Thawing (AASHTO T 103 - Procedure B)
- Soundness of Aggregate by Freezing and Thawing (AASHTO T 103 - Procedure C)
- Aggregate Durability Index (AASHTO T 210)
- Canadian Freeze-Thaw Test

Tables 38, 39, and 40 give the results for the aggregate durability index, Canadian freeze-thaw, sodium sulfate soundness, magnesium sulfate soundness test, and the three procedures of AASHTO T 103 freezing and thawing tests. The sulfate soundness tests are most commonly used in the United States. Some observations follows.

The magnesium sulfate test is more severe than the sodium sulfate test because the use of magnesium sulfate always produced higher losses than sodium sulfate. This is consistent with results from previous studies.

Some differences were observed in the evaluation of aggregate with the AASHTO T 103 freeze-thaw test and the Canadian freeze-thaw test. These differences are thought primarily to result from differences in the way aggregate

TABLE 37 British impact value and crushing value test results

Aggregate Sources	Aggregate Impact Value (% Loss)		Aggregate Crushing Value (% Loss)	
	20 - 14 mm	14 - 10 mm	20 - 14 mm	14 - 10 mm
1. Traprock, NY	**	15.4	**	16.6
2. Granite, GA	19.8	19.4	20.2	19.1
3. Steel Slag, IN	**	14.3	**	14.5
4. Gravel, MN	**	17.9	**	16.3
5. Gravel, NV	**	20.3	**	18.3
6. Limestone, IA	24.0	23.4	23.0	23.1
7. Granite, SC	31.8	31.7	32.5	33.4
8. Gravel, MN	**	18.7	**	17.0
9. Limestone, IA	**	21.2	**	19.5
10. Gravel, PA	**	19.1	**	17.6
11. Limerock, FL	33.0	32.6	30.8	32.2
12. Limestone, TX	**	24.3	**	25.2
13. Sandstone, PA	**	17.0	**	17.9
14. Limestone, MN	**	23.2	**	24.5
15. Siltstone, VA	**	11.4	**	11.5
16. Basalt, OR	**	25.8	**	26.7

** Large size materials (AASHTO 57) tested for only 4 of 16 sources.

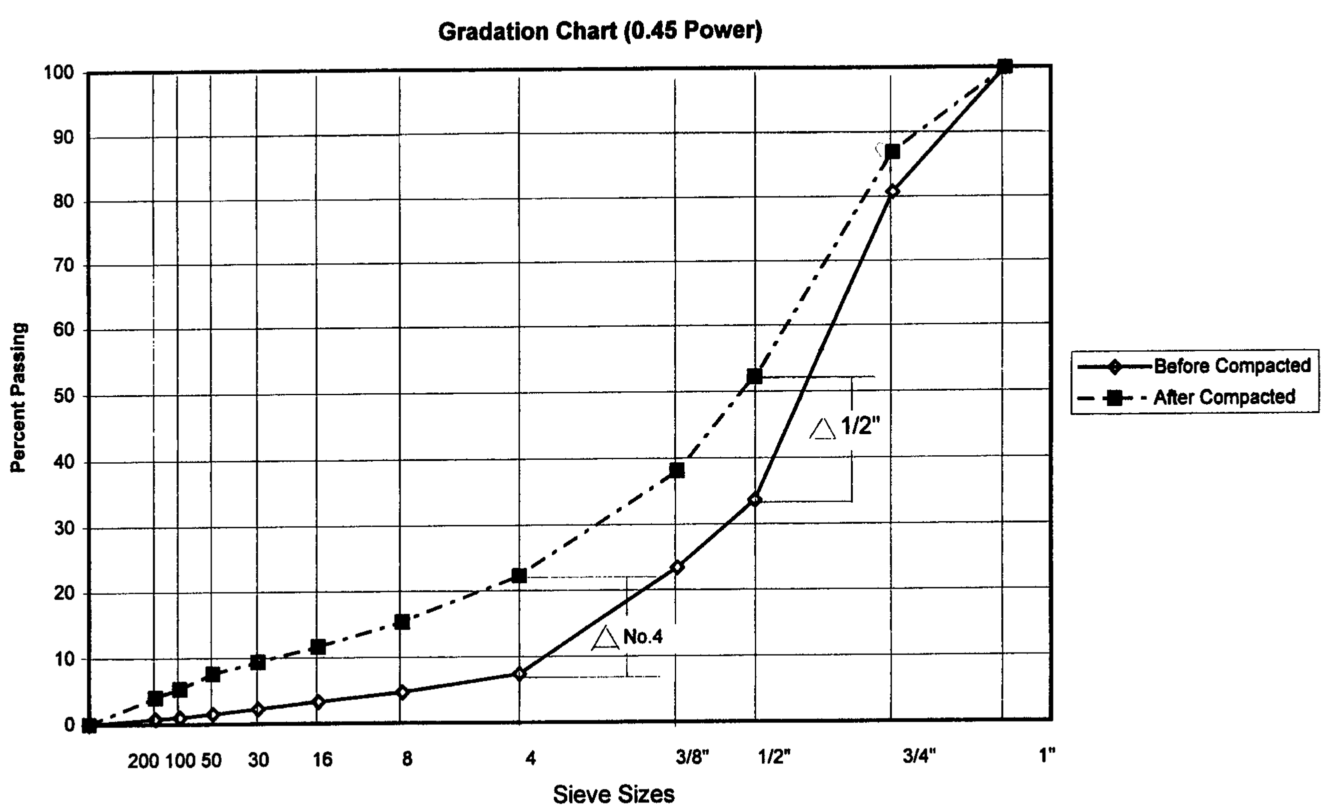
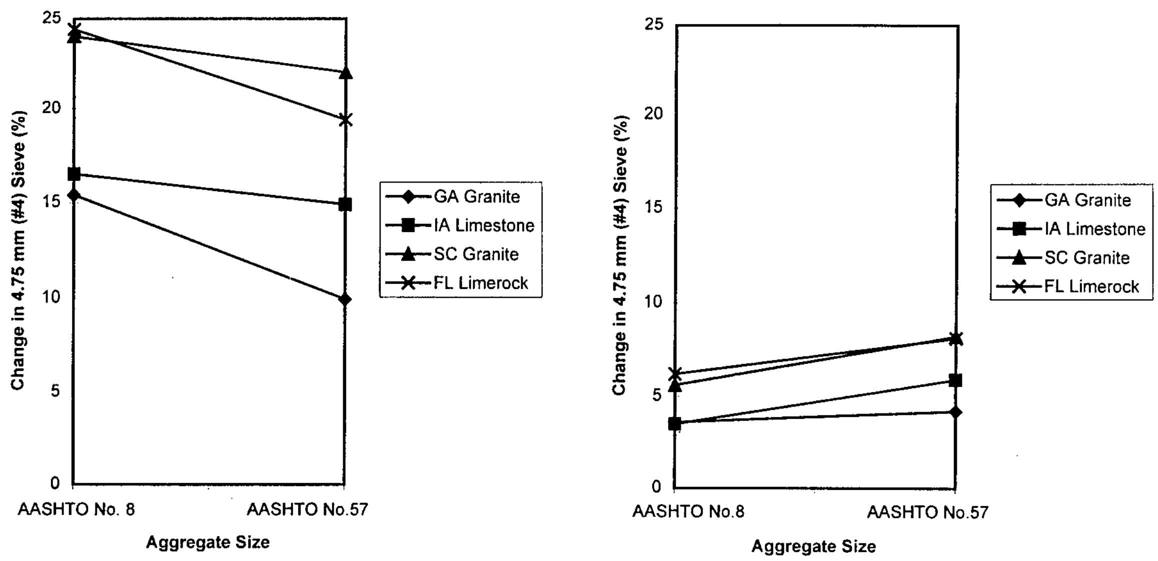


Figure 11. Computation of degradation during compaction.



(a) AASHTO No. 8 and AASHTO No. 57

(b) AASHTO No.8 + Fine and AASHTO No. 57 + Fine

Figure 12. Relationship between aggregate size and gradation changes in 4.75 mm sieve, bare aggregate.

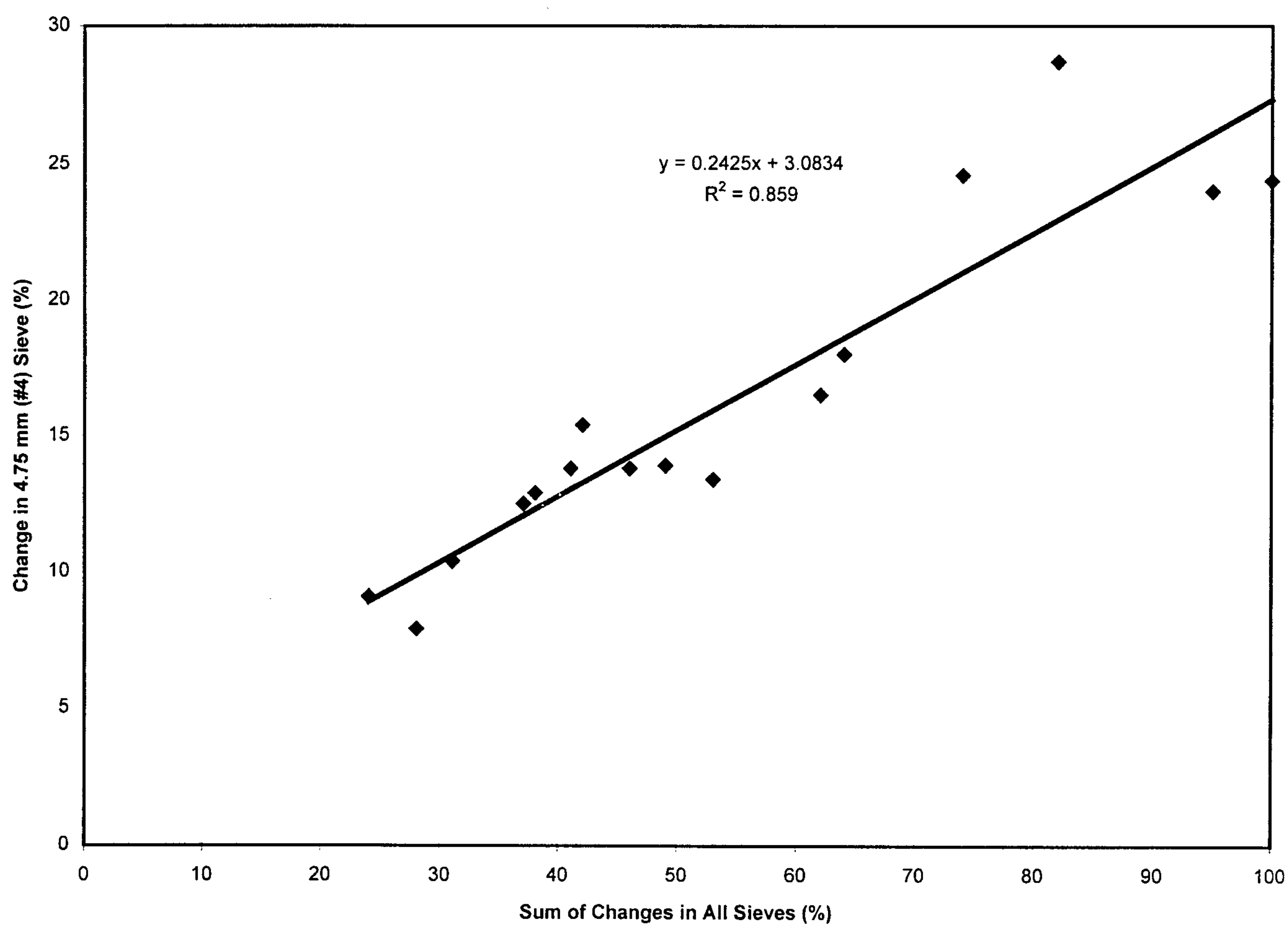


Figure 13. Relationship between sum of changes in all sieves (%) and change in 4.75 mm sieve (%), bare aggregate AASHTO 8.

TABLE 38 Aggregate durability index and Canadian freeze-thaw test results

Aggregate Sources	Aggregate Durability Index AASHTO T 210	Canadian Freeze-Thaw (% Loss)		
	19.0 - 4.75 mm	19.0 - 13.2 mm	13.2 - 9.5 mm	9.5 - 4.75 mm
1. Traprock, NY	82	2.9 (0.5)*	3.0 (0.7)	1.1 (1.0)
2. Granite, GA	85	3.4 (0.1)	0.5 (0.1)	1.6 (0.3)
3. Steel Slag, IN	73	4.6 (0.6)	1.5 (0.9)	4.5 (3.7)
4. Gravel, MN	76	7.2 (2.3)	4.5 (3.1)	5.6 (4.5)
5. Gravel, NV	66	8.2 (2.3)	4.1 (3.4)	13.5 (9.7)
6. Limestone, IA	79	4.8 (0.6)	3.0 (1.3)	1.7 (1.1)
7. Granite, SC	81	2.3 (0.1)	1.2 (0.1)	0.3 (0.3)
8. Gravel, MN	35	6.6 (3.9)	6.5 (4.3)	7.3 (5.8)
9. Limestone, IA	43	4.2 (0.4)	1.8 (0.4)	1.7 (0.6)
10. Gravel, PA	40	7.1 (3.6)	5.7 (4.1)	11.5 (8.3)
11. Limerock, FL	80	3.5 (1.3)	4.6 (2.3)	4.8 (3.8)
12. Limestone, TX	68	16.0 (6.4)	11.6 (9.0)	19.1(16.2)
13. Sandstone, PA	41	5.7 (2.1)	5.6 (2.8)	3.9 (2.4)
14. Limestone, MN	22	34.1 (23.3)	34.7 (28.4)	45.6 (38.2)
15. Siltstone, VA	71	9.9 (1.6)	5.7 (2.9)	5.8 (3.9)
16. Basalt, OR	9	20.9 (15.6)	30.1(26.6)	50.1(46.6)

* The numbers in parenthesis were calculated using "next smaller" sieves, i.e., 11.2, 8.0 and 4.0 mm.

breaks down and differences in measuring percent loss. The Canadian freeze-thaw test uses the smallest sieve size for a size fraction to measure loss. For example, the 13.2-mm sieve would be used to measure percent loss for the 19.0- to 13.2-mm fraction. The AASHTO test procedure uses the "next smallest" sieve size (i.e., 11.2 mm for the 19.0- to 13.2-mm fraction). Results using the two procedures will be similar for aggregate that degrades by abrading, flaking, or breaking into small pieces, but different when particles divide (halves, thirds, fourths, etc.). When particles divide, the Canadian procedure will indicate larger losses. Results for the Canadian freeze thaw test are shown in Table 38 for both methods of computing percent loss (i.e., using the "smallest" and "next smallest" sieves).

Petrographic Examination

The literature review revealed a consensus that petrographic examination is a valuable component of aggregate evaluations, and numerous recommendations for combining petrographic examinations with physical tests were noted. Petrographic examinations were conducted on all 16 aggregates. All petrographic reports are included in Appendix B. The standard petrographic examination report includes sample description, mineral constituents, grain size, foliation, hardness, fractures, porosity, and detailed description of the petrography. A number of researchers, including Scott (*54*), Day (*30*), and Weinert (*55*) showed that, rather than the primary minerals (sound constituents), usually secondary minerals (unsound constituents) are the principal cause of pavement problems. Various methods can be used to analyze the report including a quantitative index, called *micropetrographical index,* I_p (*56*), which is a ratio of percent sound constituents and percent unsound constituents, or percent alteration. Indexes such as this are most appropriate for igneous rocks because separation of altered from unaltered constituents in sedimentary and metamorphic rocks is, at best, difficult.

For coarse-grained igneous and metamorphic rocks there appears to be very little adverse effect on the physical properties with up to 30 percent secondary minerals (altered minerals) providing that large percentage of kaolin and/or serpentine are not present (*57*).

Sedimentary rocks, chiefly limestone and sandstone, are characterized, in general, by a lower toughness and hardness owing to the frequent presence of soft, readily cleavable

TABLE 39 Sodium sulfate soundness and magnesium sulfate soundness test results

Aggregate Sources	Sodium Sulfate (% Loss) AASHTO T 104		Magnesium Sulfate (% Loss) AASHTO T 104	
	19.0 - 9.5 mm	9.5 - 4.75 mm	19.0 - 9.5 mm	9.5 - 4.75 mm
1. Traprock, NY	**	1.7	**	1.8
2. Granite, GA	0.1	0.3	0.4	0.9
3. Steel Slag, IN	**	0.4	**	1.2
4. Gravel, MN	**	4.4	**	8.4
5. Gravel, NV	**	7.2	**	15.1
6. Limestone, IA	1.7	2.6	2.9	5.0
7. Granite, SC	0.6	0.7	2.7	3.9
8. Gravel, MN	**	9.6	**	14.9
9. Limestone, IA	**	5.6	**	9.2
10. Gravel, PA	**	12.4	**	27.9
11. Limerock, FL	13.6	12.2	30.3	43.0
12. Limestone, TX	**	12.1	**	19.3
13. Sandstone, PA	**	5.0	**	27.2
14. Limestone, MN	**	22.1	**	31.5
15. Siltstone, VA	**	0.5	**	1.7
16. Basalt, OR	**	50.6	**	59.8

** Large size materials (AASHTO 57) tested only 4 of 16 sources.

minerals such as calcites, and/or more porous structure resulting from incomplete cementation between detrital particles.

Table 41 contains source, grain size, foliation, hardness, fractures, and porosity that are rated from 1 (low level) to 4 (high level). For example, one is very fine for grain size and very low for porosity; four is very coarse for grain size and very high for porosity. The data in Table 41 are summarized from the petrographic reports. The ratings are relative and are not based on absolute or standard scales. The percent alteration minerals is reported for four igneous rocks. The last column includes a main petrographic feature that may help explain performance characterization in terms of toughness or durability. The petrographic examination indicated that three of the igneous aggregates have good toughness and durability and soundness because of their low micropetrographical index and one (basalt) has poor toughness and durability and soundness because of its high micropetrographical index. The sedimentary rocks, mostly limestones and sandstone, may soften and disintegrate because of incomplete bonding, which may be caused by wetting and drying and freezing and thawing. Hardness, fractures, and foliation are the main factors affecting toughness and abrasion resistance. Porosity and percentage of altered or secondary minerals are the main factors influencing durability and soundness.

Supplementary Test Data

In addition to toughness and abrasion resistance and soundness and durability tests, flat and elongated particles count, flakiness and elongation index, methylene blue value (performed on P200 generated in the Los Angeles abrasion test), specific gravity, and percent dust (P200) generated in the Los Angeles abrasion test (dust loss) were also measured. These test results are shown in Tables 42, 43, and 44. Although the tabulated data reveal no particular trends, several specific observations of interest can be made. The five aggregates with the largest ratio of dust loss to Los Angeles abrasion loss (Column 4 of Table 44) are four of the five carbonate crushed stones and a gravel (Source 8, Minnesota) that contains a large amount of carbonate stone. This is expected because carbonate stones are softer and less abrasion-resistant than silicates. The gravel from Nevada, Source 5, and the basalt from Oregon, Source 16, have the highest methylene blue indexes. These aggregates are composed of igneous rocks of possible volcanic nature, and the high methylene blue indexes indicate high, as expected, montmorillonite (smectite) content.

TABLE 40 Soundness by freezing-thawing (AASHTO T 103) test results

Aggregate Sources	Soundness by Freezing & Thawing (% Loss), AASHTO T103					
	Procedure A		Procedure B		Procedure C	
	19.0 -9.5 mm	9.5 - 4.75 mm	19.0 -9.5 mm	9.5 - 4.75 mm	19.0 -9.5 mm	9.5 - 4.75 mm
1. Traprock, NY	**	0.8	**	0.7	**	0.7
2. Granite, GA	0.3	0.3	0.2	0.4	0.2	0.3
3. Steel Slag, IN	**	3.8	**	3.5	**	2.2
4. Gravel, MN	**	1.6	**	8.4	**	0.6
5. Gravel, NV	**	1.4	**	2.3	**	0.6
6. Limestone, IA	1.0	1.2	1.2	1.4	0.7	0.8
7. Granite, SC	0.3	0.4	0.1	0.2	0.8	0.1
8. Gravel, MN	**	4.3	**	13.4	**	2.4
9. Limestone, IA	**	2.4	**	1.2	**	1.4
10. Gravel, PA	**	4.5	**	25.7	**	0.8
11. Limerock, FL	3.8	14.6	2.3	2.8	1.7	8.1
12. Limestone, TX	**	8.5	**	1.8	**	2.0
13. Sandstone, PA	**	2.0	**	2.8	**	1.4
14. Limestone, MN	**	15.0	**	49.1	**	12.2
15. Siltstone, VA	**	0.8	**	1.3	**	0.6
16. Basalt, OR	**	15.7	**	24.8	**	11.0

** Large size materials (AASHTO 57) tested only 4 of 16 sources.

Field Data

Field data on the generation of fines during mix production and placement were collected and analyzed for three of the aggregate sources included in the toughness and abrasion resistance and durability and soundness portion of the study. These were Source 4 (Minnesota Gravel), Source 5 (Nevada Gravel) and Source 6 (Florida Limerock). The data included stockpile, cold feed, mix, and mix from cores gradations.

Analysis of field data indicated that, as expected, some aggregate degradation occurs during mix production and placement. However, in all three cases, acceptable levels of dust (P200) were maintained. Baghouses were used in two cases and the amount of baghouse fines returned to the mix was controlled. In the third, the plant was equipped with a wet scrubber and no captured fines were returned to the mix. Because these data were limited and more than one aggregate type was used in HMA production, it was not possible to correlate generation of fines with aggregate tests or performance.

Relationship Between Aggregate Test Results

Using correlation analysis (SAS Program), correlation matrixes among aggregate properties were developed. A correlation matrix for toughness and abrasion resistance is shown in Table 45; a correlation matrix for durability and soundness is shown in Table 46. The Micro-Deval test was included in both groups because it is a combination of abrasion (steel balls) and weathering (water added) to see if there is any favorable correlation for one or both groups. The correlation coefficients (R values) are the top numbers in each cell. The bottom numbers in each cell are the statistical significance levels (P values) corresponding to the correlation coefficients.

The following observations can be made from the correlations:

- Los Angeles abrasion has good correlation with aggregate impact value, aggregate crushing value, and Superpave gyratory compactor degradation of both open- and

TABLE 41 Summary of petrographic examination

Rock Type and State	Grain size	Foliation	Hardness	Fractures	Porosity	% Alteration	Petrographic Feature
1. Traprock, NY	3	1	4	1	1	10	Some alterations but does not weaken because they are restricted to small locations
2. Granite, GA	3	4	3	2	2	4	Minor alternations
3. Steel Slag, IN	1	1	2	1	3	_	"Air bubbles" provide some porosity
4. Gravel, MN	1-2 *	1-2	1-4	1-2	1-2	tr**	Very minor alteration of feldspars in each rock type
5. Gravel, NV	1	1	2	2	3	_	Fractures filled with finely-grained mica
6. Limestone, IA	2	1	1	1	3	_	Well-cemented dolomite, more resistant to dissolution than calcite in most of others
7. Granite, SC	4	4	4	3	1	tr**	Penetrative foliation, coarse grain size results in a susceptibility to breakage
8. Gravel, MN	2	1-3	1-3	1-2	1-2	_	Intermediate proportions of soft limestone
9. Limestone, IA	2	1	1	1	2	_	Carbonate mud account for 10 percent
10. Gravel, PA	3	1	2	1	1	_	Well indurated sandstone, with lower proportion of mica than the other sandstone
11. Limerock, FL	2	1	1	1	4	_	Finely-grained calcite mud occurs with skeletal remains
12. Limestone, TX	3	1	1	1	1	_	Solubility of calcite and relatively low hardness
13. Sandstone, PA	3	1	2	1	1	_	Abundance of finely-grained mica that are disseminated throughout the rock, coating most quartz grains and preventing strong cementation
14. Limestone, MN	3	1	1	1	1	_	Limestone supported with finely-grained calcite mud and with 10 to 40% skeletal fragments
15. Siltstone, VA	1	4	3	3	1	_	Slate composed of insoluble, well-cemented materials
16. Basalt, OR	1	1	1	2	2	50	Fine grained highly weathered basalt

*The rating shown here is a range which is based on the ratings of the igneous, sedimentary and metamorphic rocks in the gravel.

**tr= trace of alteration minerals

dense-graded bare aggregate. Incorporation of asphalt cement reduces the strength of the correlations between Los Angeles abrasion and Superpave gyratory compactor aggregate degradation.

- Correlations between Los Angeles abrasion and Superpave gyratory compactor degradation are stronger for open-graded than for dense-graded aggregate.
- The Micro-Deval is better correlated with durability and soundness tests than with toughness and abrasion resistance tests. It has no good or fair correlation with any toughness and abrasion resistance tests, but has a good correlation with magnesium sulfate soundness and fair correlation with sodium sulfate soundness and durability index.
- The aggregate durability index test did not have a good correlation with any of the other durability and soundness tests. This is thought indicative of the specialized nature of the test (i.e., it was developed for basalt aggregates indigenous to northwestern states).

Large Stone Versus Small Stone

A limited investigation was made of differences in degradation of small stone (AASHTO 8, maximum nominal size of 9.5 mm) and large stone (AASHTO 57, maximum nominal size of 19 mm) using the gyratory compactor. Large size stone was tested for only 4 of the 6 total sources. As discussed previously, the gradation change in the 4.75 mm (No. 4) sieve is the best choice to measure the degradation without bias because of the aggregate size, which means that it is not sensitive to aggregate size. Degradation amounts (the gradation change in 4.75 mm [No. 4] sieve) were evaluated for the four aggregate types (*58*). A typical plot representing South Carolina granite is shown in Figure 14. The plots indicate that the open-graded aggregate degraded more than the dense-graded aggregate for both large and small sizes and that the presence of asphalt cement binder generally decreases aggregate degradation.

TABLE 42 Specific gravity and absorption test data

Aggregate Sources	Specific Gravities and Absorptions			
	Bulk Specific Gravity	SSD Bulk specific Gravity	Apparent Specific Gravity	Absorption (%)
1. Traprock, NY	2.886	2.918	2.981	1.1
2. Granite, GA	2.678	2.704	2.749	1.0
3. Steel Slag, IN	3.252	3.317	3.478	2.0
4. Gravel, MN	2.683	2.714	2.773	1.2
5. Gravel, NV	2.553	2.596	2.702	2.5
6. Limestone, IA	2.604	2.664	2.770	2.3
7. Granite, SC	2.633	2.658	2.702	1.0
8. Gravel, MN	2.604	2.656	2.748	2.1
9. Limestone, IA	2.577	2.639	2.748	2.4
10. Gravel, PA	2.551	2.602	2.687	2.0
11. Limerock, FL	2.349	2.449	2.612	4.3
12. Limestone, TX	2.599	2.645	2.726	1.8
13. Sandstone, PA	2.639	2.682	2.760	1.6
14. Limestone, MN	2.582	2.632	2.717	1.9
15. Siltstone, VA	2.694	2.717	2.758	0.9
16. Basalt, OR	2.485	2.615	2.857	5.2

* The specific gravity is calculated using the following gradation:

Size	% of total sample weight
19.0-12.5 mm	50
12.5-9.5 mm	25
9.5-4.75 mm	25

Particle Shapes and Degradation

Correlations between flat and elongated particle shape and toughness and abrasion tests were investigated because it was postulated that more flat and elongated particles would be more susceptible than equidimensional particles to breakage and abrasion. However, the Los Angeles abrasion, aggregate impact, aggregate crushing, and Superpave gyratory compactor degradation test data indicate no correlation. This lack of correlation does not necessarily mean a relationship might not exist between toughness and abrasion resistance and particle shape. It may mean that, as studied, this influence could not be separated from the more important effects of intrinsic rock strength and abrasion resistance. A more revealing study would require separation of particles of several rock types according to shape (ASTM proportional calipers or British flakiness and elongation template) and testing and comparing of the various groups.

Relationship Between Aggregate Test Results and Pavement Performance

Contacts were made with state DOTs to select the 16 aggregates used in this study. Initial general pavement performance ratings listed in Table 47 were developed from these contacts. The following preliminary guidelines as used by Senior and Rogers (*39*) were used to identify good, fair, and poor HMA pavement performance:

- Good—Used for many years with no significant aggregate degradation problem during construction and no significant popouts, raveling, or potholes during service life.
- Fair—Used at least once where some degradation occurred during construction and some popouts, raveling, and potholes developed, but pavement life extended for over 8 years.

TABLE 43 Aggregate particle shapes

Aggregates Sources	Flat and Elongated Particles (ASTM D4791)*			Flakiness/Elongation Index (BS 812, Part 105)*	
	1:2	1:3	1:5	Flakiness	Elongation
1. Traprock, NY	39	13	1	22	28
2. Granite, GA	57	19	1	36	16
3. Steel Slag, IN	12	2	0	5	15
4. Gravel, MN	22	4	0	15	14
5. Gravel, NV	17	3	0	11	13
6. Limestone, IA	14	2	0	7	17
7. Granite, SC	37	16	1	18	33
8. Gravel, MN	12	1	0	7	19
9. Limestone, IA	30	4	0	15	9
10. Gravel, PA	46	20	2	36	24
11. Limerock, FL	32	7	0	12	17
12. Limestone, TX	35	1	0	15	22
13. Sandstone, PA	47	11	1	22	40
14. Limestone, MN	38	7	1	23	21
15. Siltstone, VA	58	28	5	37	42
16. Basalt, OR	22	5	0	10	17

* The numbers shown here are weighted average percentages based on the following gradation:

Size for ASTM D4791	Size for BS 812	% of total sample weight
19.0-12.5 mm	20mm - 14mm	50
12.5-9.5 mm	14mm - 10mm	25
9.5-4.75 mm	10mm - 6.3mm	25

TABLE 44 Analysis of dust from Los Angeles abrasion test

Aggregate	LA Abrasion Loss (%)*	Dust (-#200) Loss (%)	Ratio (%) (Dust Loss/LA Loss)	Methylene Blue Index (mg MB / gm dust)
1. Traprock, NY	17.4	5.4	31.0	7.0
2. Granite, GA	23.9	6.3	26.3	2.0
3. Steel Slag, IN	14.5	4.6	31.7	0.5
4. Gravel, MN	21.2	7.5	35.5	6.0
5. Gravel, NV	19.5	6.6	33.8	19.0
6. Limestone, IA	30.0	14.2	47.3	2.0
7. Granite, SC	50.3	14.9	29.6	1.8
8. Gravel, MN	24.9	9.7	39.1	11.3
9. Limestone, IA	27.1	11.1	41.0	4.3
10. Gravel, PA	26.2	9.3	35.5	5.0
11. Limerock, FL	42.0	17.9	42.6	1.5
12. Limestone, TX	26.9	9.0	33.5	10.3
13. Sandstone, PA	25.5	8.7	33.9	5.8
14. Limestone, MN	26.9	10.9	40.5	8.0
15. Siltstone, VA	13.6	4.0	29.3	3.8
16. Basalt, OR	31.9	8.6	27.0	22

* LA Losses reported here are single test results from the retest to determine the amount of dust and are slightly different from the values reported in Table 6.4 (average of three replicates).

TABLE 45 Correlation matrix for toughness and abrasion resistance[a]

	LA Abrasion	Aggregate Impact Value	Aggregate Crushing Value	Superpave Compactor AASHTO8 Open Graded Bare Aggregate	Superpave Compactor AASHTO8 Open Graded Mix	Superpave Compactor AASHTO8 Dense Graded Bare Aggregate	Superpave Compactor AASHTO8 Dense Graded Mix	Micro-Deval Abrasion
LA Abrasion	1.00000 0.0	0.93214 0.0001	0.93445 0.0001	0.79557 0.002	0.81283 0.0001	0.74103 0.0010	0.55490 0.0257	0.39096 0.1343
Aggregate Impact Value		1.00000 0.0	0.98052 0.0001	0.83396 0.0001	0.85602 0.0001	0.77811 0.0004	0.59884 0.0142	0.34030 0.1972
Aggregate Crushing Value			1.00000 0.0	0.87643 0.0001	0.86729 0.0001	0.81505 0.0001	0.57460 0.0199	0.35912 0.1719
Superpave Compactor AASHTO8 Open Graded Bare Aggregate				1.00000 0.0	0.89672 0.0001	0.93964 0.0001	0.58470 0.0174	0.52436 0.0371
Superpave Compactor AASHTO8 Open Graded Mix					1.00000 0.0	0.94175 0.0001	0.76797 0.0005	0.48981 0.0541
Superpave Compactor AASHTO8 Dense Graded Bare Aggregate						1.00000 0.0	0.72944 0.0013	0.40632 0.1184
Superpave Compactor AASHTO8 Dense Graded Mix							1.00000 0.0	0.14270 0.5980
Micro-Deval Abrasion								1.00000 0.0

[a]Top values are correlation coefficients R and bottom values are significance levels P in each cell.

TABLE 46 Correlation matrix for soundness and durability[a]

	Sodium Sulfate	Magnesium Sulfate	Freeze-Thaw Pro. A	Freeze-Thaw Pro. B	Freeze-Thaw Pro. C	Canadian Freeze-Thaw	Modified Canadian Freeze-Thaw	Durability Index	Micro-Deval Abrasion
Sodium Sulfate	1.00000 0.0	0.87804 0.0001	0.80908 0.0001	0.64025 0.0075	0.80342 0.0002	0.83405 0.0001	0.87325 0.0001	-0.75911 0.0006	0.69229 0.0030
Magnesium Sulfate		1.00000 0.0	0.85875 0.0001	0.55880 0.0244	0.79521 0.0002	0.67324 0.0043	0.70486 0.0023	-0.68115 0.0037	0.84813 0.0001
Freeze-Thaw Pro. A			1.00000 0.0	0.64525 0.0069	0.94544 0.0001	0.78147 0.0004	0.79609 0.0002	-0.55194 0.0266	0.61676 0.0109
Freeze-Thaw Pro. B				1.00000 0.0	0.72721 0.0014	0.83540 0.0001	0.83231 0.0001	-0.74440 0.0009	0.61766 0.0108
Freeze-Thaw Pro. C					1.00000 0.0	0.84093 0.0001	0.85732 0.0001	-0.60924 0.0030	0.57787 0.0030
Canadian Freeze-Thaw						1.00000 0.0	0.99526 0.0001	-0.72723 0.0014	0.59110 0.0159
Modified Canadian Freeze-Thaw							1.00000 0.0	-0.74245 0.0010	0.60900 0.0123
Durability Index								1.00000 0.0	-0.74358 0.0010
Micro-Deval Abrasion									1.00000 0.0

[a] Top values are correlation coefficients R and bottom values are significance levels P in each cell.

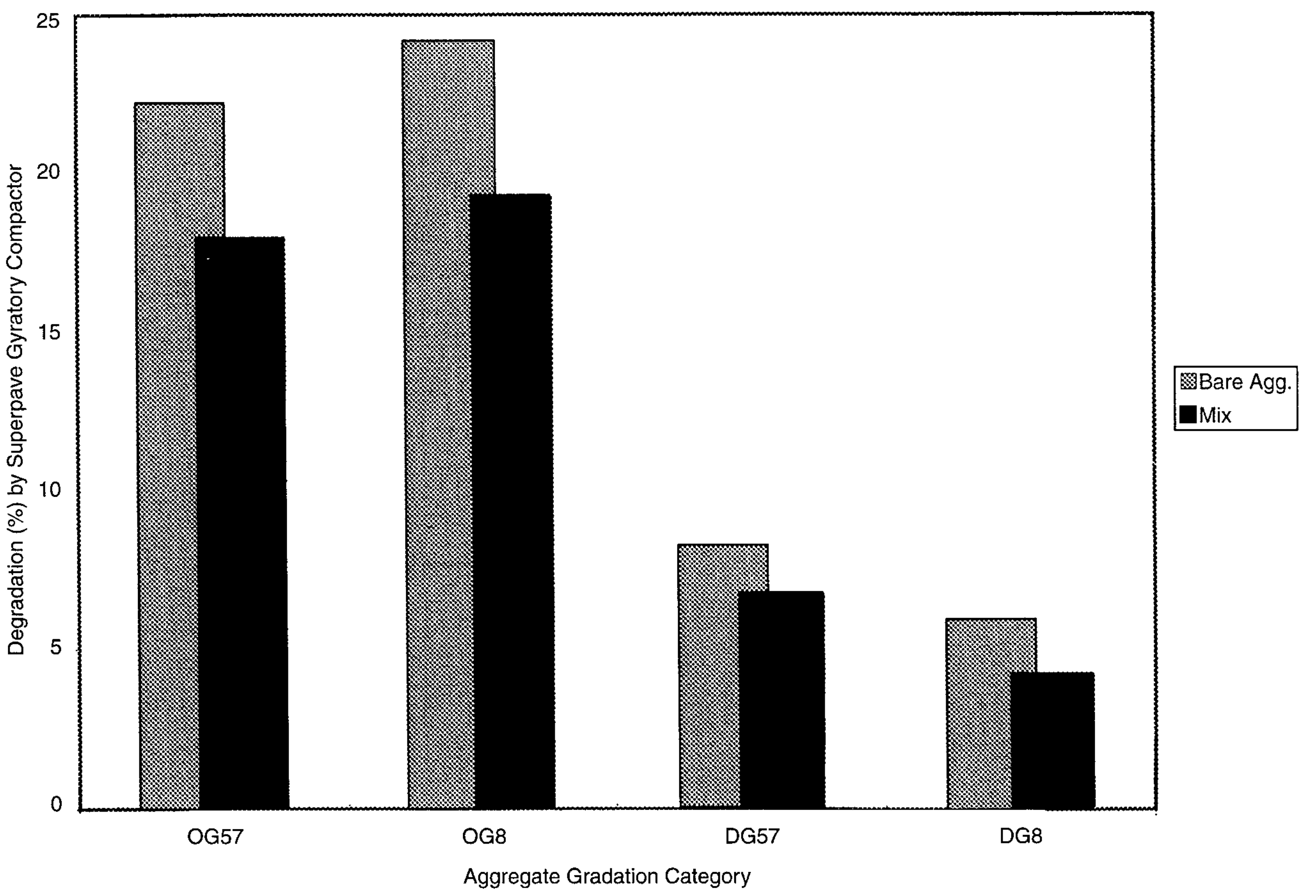


Figure 14. Degradation (%) in various gradation categories, Granite, SC.

- Poor—Used at least once where raveling, popouts, or combinations thereof developed during the first 2 years, severely restricting pavement life.

Additional data were collected to refine the pavement performance rating. Pavement performance evaluation sheets were sent to agencies with fair or poor performing aggregates to collect the information related to pavement distress modes, aggregate-related causes of distress, and supplementary information that could help evaluate the performance of aggregates in asphalt concrete pavement. The completed pavement performance evaluations for aggregate with fair and poor performance are included in Appendix C. Visits were made to several states to observe pavement conditions and discuss performance with DOT materials personnel. On the basis of these additional data, each aggregate was rated independently in terms of both toughness and abrasion resistance and soundness and durability. These ratings are also shown in Table 47. The "worst case" of these ratings are tabulated in Table 47 as indicators of overall pavement performance.

Tables 48 and 49 represent a second approach to establish performance. Test data were compared with criteria discovered in the literature review and qualitative performance ratings were assigned. The overall expected performance ratings assigned to the sources generally confirm the ratings in Table 47. The reason for only a few sources with poor expected performance is thought to be that all sources, except the Oregon basalt, are used by states and, therefore, meet their specifications. The review of state specifications revealed considerable consistency in acceptance criteria.

Source 10, Pennsylvania gravel, and source 15, Virginia siltstone, illustrate the difficulties encountered in establishing reliable indications of pavement performance. Source 10 was selected on the basis of expected fair performance. However, after testing was completed and analyses started the characterization as a fair performer became questionable. Several projects were identified that had required sealing within 4 years because of popouts and raveling.

Source 15 was initially selected on the basis of expected poor performance. However, all test results, both toughness and abrasion resistance and durability and soundness, indicated pavement constructed using the aggregate in asphalt concrete should perform well. A site visit and conversations with Virginia DOT field personnel indicated that pavements constructed with the aggregate did indeed frequently perform poorly, but not because of deficiencies in aggregate toughness, abrasion resistance, durability, or soundness. Asphalt

TABLE 47 Pavement performance rating for aggregate sources

Rock Type and State	Initial General Rating for Source Selection	Performance Rating Based on Toughness / Abrasion	Performance Rating Based on Durability / Soundness	Overall (worst case) Performance Rating
1. Traprock, NY	G	G	G	G
2. Granite, GA	G	G	G	G
3. Steel Slag, IN	G	G	G	G
4. Gravel, MN	G	G	G	G
5. Gravel, NV	G	G	G	G
6. Limestone, IA	G	G	G	G
7. Granite, SC	F	F	G	F
8. Gravel, MN	F	F	F	F
9. Limestone, IA	F	F	G	F
10. Gravel, PA	F	P	P	P
11. Limerock, FL	P	P	N*	P
12. Limestone, TX	P	P	P	P
13. Sandstone, PA	P	P	P	P
14. Limestone, MN	P	N**	P	P
15. Siltstone, VA	P	N	N	N
16. Basalt, OR	P	N**	P	P

Notes:
Ten of sixteen aggregates are from SHRP Wet-Freeze Climatic Region.
G Good pavement performance
F Fair pavement performance
P Poor pavement performance
N Not a factor in assessing pavement performance
* Test results compared with criteria for several durability/soundness tests indicate fair performance might be expected.
** Test results compared with criteria for several toughness/abrasion resistence tests indicate fair performance might be expected.

pavements constructed with this aggregate have bleeding and rutting problems. Therefore, source 15 was excluded from the analyses.

The evaluation of field performance of pavements in this study indicates that toughness and abrasion are not a dominant factor directly affecting asphalt concrete, but may be a factor during construction where degradation can occur which will ultimately affect pavement performance. This is consistent with a study reported by Brown (*59*) where cracked aggregate did not appear to have affected the performance of chipping rolled asphalt as part of the structure as long as the stone remained in the asphalt surface. Goode and Owing (*60*) report a 13-year study of dense-graded asphalt concrete where only minor degradation of the aggregate occurred during pavement service and this did not affect the pavement performance. Rogers and Senior (*61*) report that granite gneiss found in the Precambrian shield areas of Ontario with LA abrasion losses up to 55 percent performed satisfactorily in asphalt, granular base, and concrete, provided precautions are taken to prevent excessive handling prior to placement. The key to using aggregate with marginal toughness and abrasion resistance is construction control. This is particularly important for fines that may be generated during mix production and cracking and crushing that may occur during compaction. Dust generation can be controlled by adjusting mix designs (using aggregate with less dust) or by wasting baghouse fines. Aggregate cracking and crushing during compaction are more difficult to control, unless gradation of cores is part of the construction management process.

On the other hand, evaluation of field performance demonstrates that aggregate durability and soundness are significant factors that directly affect pavement performance. Soundness and durability are believed to be associated with degradation occurring upon exposure to environmental factors such as wetting and drying or freezing and thawing. Literature reviewed (*26, 29, 30, 54*) indicates that durability and soundness of aggregate results directly in pavement distress. The rock types identified in most of these studies were basalt, sandstone, and shale. A pavement constructed with aggregates that have large percentages of altered minerals, poor cementation, or soluble calcite can deteriorate with cycles of

TABLE 48 Comparison of toughness/abrasion test results and criteria

Test	Micro-Deval Abrasion (1)	Los Angeles Abrasion (2)	Aggregate Impact Value (3)	Aggregate Crushing Value (3)	
Limits	Max. 17 Surface Max. 20 Binder	Max. 40-45	Max. 30	Max. 30	Overall Expected Performance
	Mea. Value/ Expected Per.	Mea. Value/ Expected Perf.	Mea. Value/ Expected Per.	Mea. Value/ Expected Perf.	
1. Traprock, NY	7.1/Good	17.4/Good	15.4/Good	16.6/Good	Good
2. Granite, GA	6.5/Good	24.5/Good	19.4/Good	19.1/Good	Good
3. Steel Slag, IN	9.9/Good	14.5/Good	14.3/Good	14.5/Good	Good
4. Gravel, MN	9.6/Good	21.2/Good	17.9/Good	16.3/Good	Good
5. Gravel, NV	15.2/Fair	19.5/Good	20.3/Good	18.3/Good	Good
6. Limestone, IA	15.0/Fair	30.0/Good	23.4/Fair	23.1/Fair	Fair
7. Granite, SC	15.6/Fair	49.0/Poor	31.7/Poor	33.4/Poor	Poor
8. Gravel, MN	14.6/Fair	25.5/Good	19.1/Good	17.6/Good	Good
9. Limestone, IA	13.0/Good	25.0/Good	18.7/Good	17.0/Good	Good
10. Gravel, PA	14.6/Fair	26.4/Good	21.2/Good	19.5/Good	Good
11. Limerock, FL	13.0/Good	25.0/Good	18.7/Good	17.0/Good	Good
12. Limestone, TX	18.1/Poor	27.4/Good	24.3/Fair	25.2/Fair	Fair
13. Sandstone, PA	34.0/Poor	27.6/Good	17.0/Good	17.9/Good	Fair
14. Limestone, MN	28.9/Poor	28.3/Good	23.2/Fair	24.5/Fair	Fair
15. Siltstone, VA	8.0/Good	12.9/Good	11.4/Good	11.5/Good	Good
16. Basalt, OR	35.2/Poor	31.9/Good	25.8/Fair	26.7/Fair	Fair

Notes:

(1) Limits recommended in specifications of Ontario Ministry of Transportation, Canada.
(2) Approximately 80% of states specify limits within this range.
(3) Limits recommended in British Specification VS 812.

wetting and drying or freezing and thawing. Lost aggregate strength and asphalt concrete stability can result in popouts, raveling, cracking, and, in severe cases, potholing. According to performance evaluation reports, pavements constructed with poor performing aggregates had aggregate popouts, raveling, and potholes.

Toughness and Abrasion Resistance Tests Versus Performance

Relationships between asphalt concrete pavement performance and toughness and abrasion resistance test results are presented in Figures 15 through 19. Sources 14, 15, and 16 are also included in the plots, but not considered in the analyses, as noted in Table 47. Each of the three sources is shown in an open triangle (△).

These plots indicate that no test clearly and distinctly delineates performance of all aggregates. The Micro-Deval, Figure 19, seems to provide the best performance predictability. The three solid horizontal lines at the average for each performance group are in the proper order. The dashed horizontal line at a loss of 18 percent separates poor from fair and good performing aggregate.

Aggregate number seven, South Carolina granite, exhibited inconsistent response. The granite is coarse grained and brittle. Losses for the LA, impact, crushing, and gyratory compaction tests were large and indicative of poor performance. However, the Micro-Deval, which is primarily abrasive, has smaller losses and is indicative of fair to good performance. The performance evaluation indicates the sole problem with this aggregate is control of fines generation during production.

TABLE 49 Comparison of durability/soundness test results and criteria

Test	Aggregate Durability Index(1)	Canadian Unconfined Freeze-Thaw(2)	Sodium Sulfate(3)	Magnesium Sulfate(4)	AASHTO Freeze-Thaw(5)	Over all Expected Performance
Limits	Min. 30-40	Max. 6 Surface Max. 15 Binder	Max. 11-15	Max. 11-20	Max. 25 Pro. A Max. 30 Pro. B Max. 10 Pro. C	
	Mea. Value/ Expected Perf.	Mea. Value/ Expected Perf.	Mea. Value/ Expected Perf.	Mea. Value/ Expected Perf.	Mea. Value/ Expected Perf.	
1. Traprock, NY	82/Good	2.4/Good	1.7/Good	1.8/Good	0.8(A), 0.7 (B), 0.7/C/Good	Good
2. Granite, GA	85/Good	1.9/Good	0.3/Good	0.9/Good	0.3(A), 0.4(B), 0.3(C)/Good	Good
3. Steel Slag, IN	73/Good	3.5/Good	0.4/Good	1.2/Good	3.8(A), 3.5(B), 3.2(C)/Good	Good
4. Gravel, MN	76/Good	5.8/Good	4.4/Good	8.4/Good	1.6(A), 8.4(B), 0.6(C)/Good	Good
5. Gravel, NV	66/Good	8.5/Fair	7.2/Good	15.1/Fair	1.4(A), 2.3(B), 0.6(C)/Good	Good
6. Limestone, IA	79/Good	3.2/Good	2.6/Good	5.0/Good	1.2(A), 1.4(B), 0.8(C),/Good	Good
7. Granite, SC	81/Good	1.3/Good	0.7/Good	3.9/Good	0.4(A), 0.2(B), 0.1(C)/Good	Good
8. Gravel, PA	40/Poor	8.0/Fair	12.4/Fair	27.9/Fair	4.3(A), 13.4(B), 2.4(C)/Fair	Poor
9. Gravel, MN	35/Poor	6.8/Fair	9.6/Good	14.9/Fair	4.3(A), 13.4(B), 2.4(C)/Good	Fair
10. Limestone, IA	43/Fair	2.6/Good	5.6/Good	9.7/Good	2.4(A), 1.2(B), 1.4(C)/Good	Good
11. Limerock, FL	80/Good	4.3/Good	12.2/Fair	43.0/Poor	14.6(A), 2.8(B), 8.1(C)/Fair	Fair
12. Limestone, TX	68/Good	15.5/Poor	12.1/Fair	19.3/Fair	8.5(A), 1.8(B), 8.1(C)/Fair	Fair
13. Sandstone, PA	41/Fair	5.1/Good	5.0/Good	27.2/Poor	2.0(A), 2.8(B), 1.4(C)/Good	Fair
14. Limestone, MN	22/Poor	38.0/Poor	22.1/Poor	31.5/Poor	15.0(A), 49.1(B), 12.2(C)/Poor	Poor
15. Siltstone, VA	71/Good	7.2/Fair	0.5/Good	1.7/Good	0.8(A), 1.3(B), 0.6(C)/Good	Good
16. Basalt, OR	9/Poor	33.3/Poor	50.6/Poor	59.8/Poor	15.7(A), 24.8(B), 11.9(C)/Poor	Poor

Notes:
(1) Limits based on five state specifications.
(2) Limits recommended in specifications of Ontario Ministry of Transportation, Canada.
(3) Approximately 60% of states using sodium sulfate soundness specify limits within this range.
(4) Approximately 75% of states using magnesium sulfate soundness specify limits within this range.
(5) Only 5 of the states surveyed have criteria from some freeze-thaw tests. Limits for procedures A and C are from Iowa DOT Specification, and limit for procedure B is from Nebraska DOT specification.

Rock Source	Description	Rock Source	Description	Rock Source	Description	Rock Source	Description
1	Traprock,NY	5	Gravel,NV	9	Limestone, IA	13	Sandstone,PA
2	Granite,GA	6	Limestone,IA	10	Gravel, PA	14	Limestone,MN
3	Steel Slag, IN	7	Granite,SC	11	Limerock,FL	15	Siltstone,VA
4	Gravel,MN	8	Gravel, MN	12	Limestone,TX	16	Basalt,OR

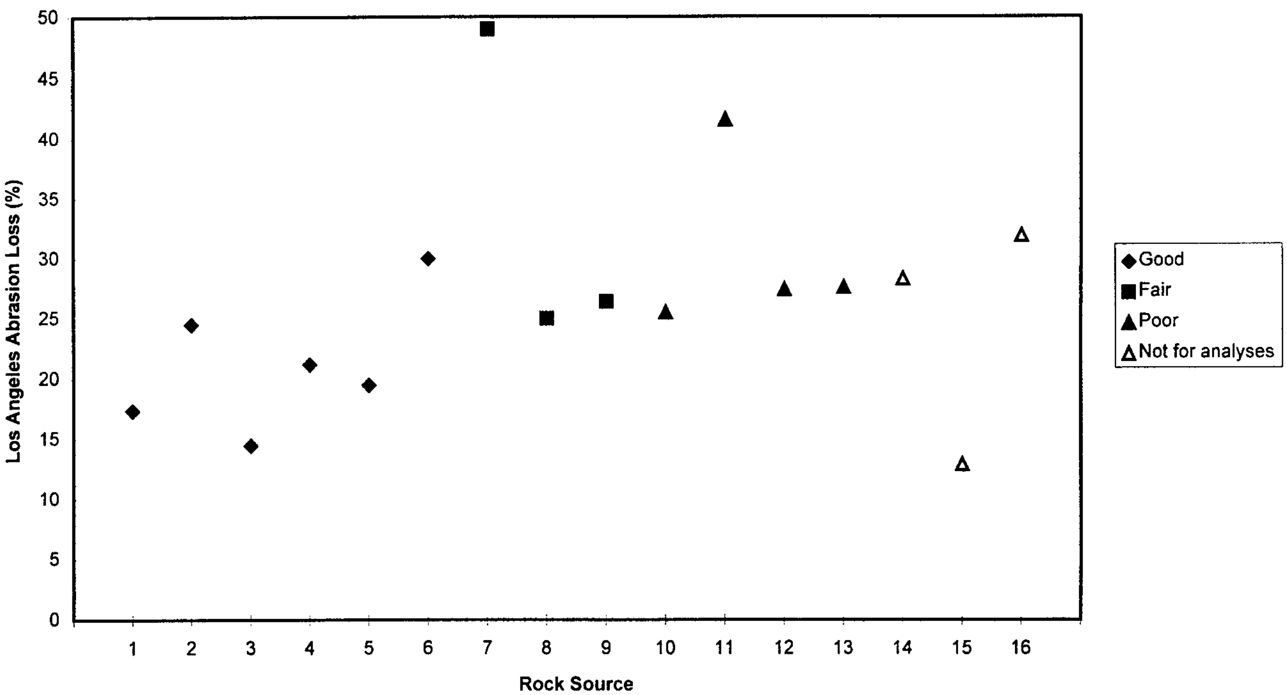


Figure 15. Pavement performance ratings and Los Angeles abrasion loss.

Senior and Rogers (*39*) reported good to fair performance predictability with the Micro-Deval test. They found Micro-Deval loss of 20 percent separates good to fair aggregate from poor. The Ontario Provincial Standard Specification, Ministry of Transportation, Ontario, Canada (*62*) specifies maximum Micro-Deval loss of 20 percent for a binder mix and 17 percent for a surface mix.

Durability and Soundness Tests Versus Performance

Relationships between pavement performance and some durability and soundness test results are presented in Figures 20 through 23. Sources 11 and 15 are also included in the plots, but not considered in the analyses, as noted in Table 47. Each of the two sources is shown in an open triangle (△).

Micro-Deval abrasion is also included in plots because it is a wet-abrasion test and may provide some indication of aggregate "weathering" susceptibility. These plots, and other plots for AASHTO T 103 (all three procedures) and Canadian Freeze and Thaw not shown here, again indicate that no test clearly and distinctly delineates performance of all aggregates. However, the magnesium sulfate soundness and Micro-Deval tests indicate fair predictability. The three solid horizontal lines in Figures 22 and 23 at the average for each performance group are in the proper order. The dashed horizontal lines at losses of 18 percent, for both the magnesium sulfate and Micro-Deval tests, separate poor from fair and good performing aggregate.

As noted previously, Senior and Rogers (*39*) found good to fair predictability for the Micro-Deval test with 20 percent loss separating good to fair aggregate from poor. Paul (*33*) indicates portland cement concrete pavements with fine aggregate having magnesium sulfate soundness losses greater than 20 percent performed poorly and those with losses less than 17 percent performed well. Papaleontiou et al. (*36*) and Hasan et al. (*37*) recommended a combination magnesium sulfate soundness and wet ball mill (Texas degradation) criteria for aggregate quality.

Statistical Analysis

Regression analyses were made to develop equations for predicting pavement performance with those physical aggre-

Rock Source	Description	Rock Source	Description	Rock Source	Description	Rock Source	Description
1	Traprock,NY	5	Gravel,NV	9	Limestone, IA	13	Sandstone,PA
2	Granite,GA	6	Limestone,IA	10	Gravel, PA	14	Limestone,MN
3	Steel Slag, IN	7	Granite,SC	11	Limerock,FL	15	Siltstone,VA
4	Gravel,MN	8	Gravel, MN	12	Limestone,TX	16	Basalt,OR

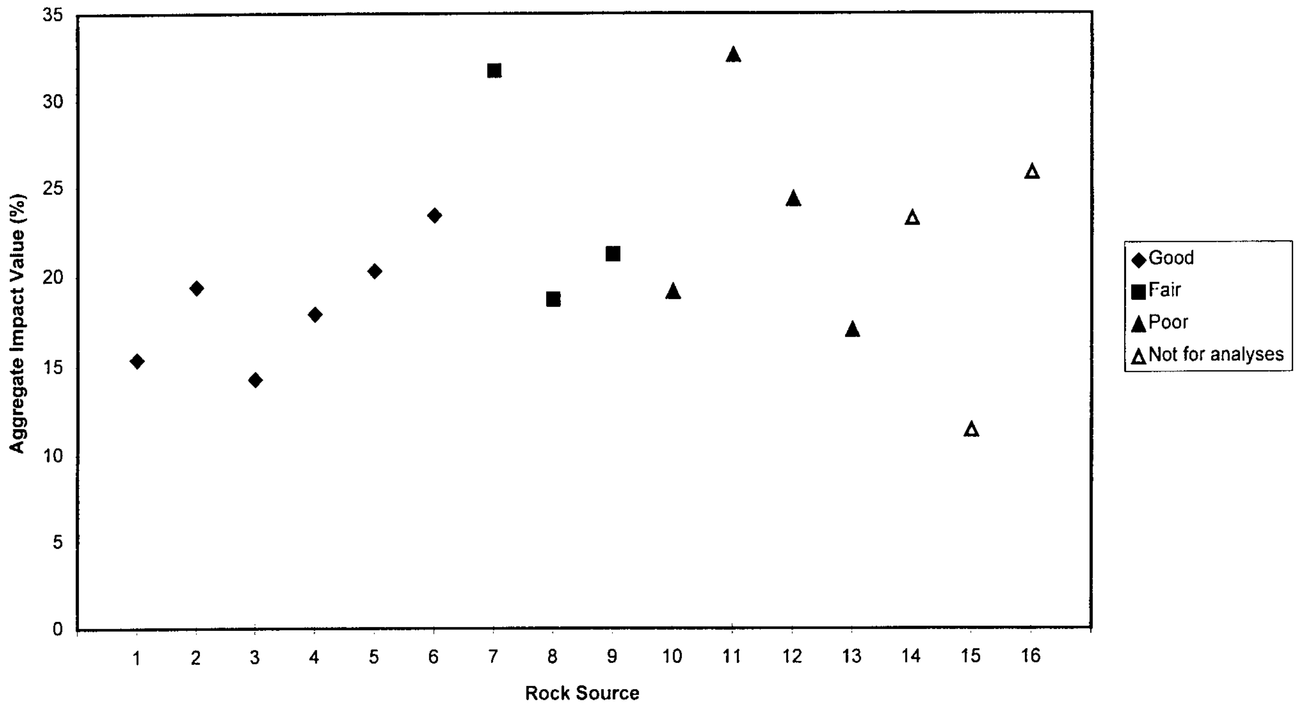


Figure 16. Pavement performance ratings and aggregate impact value.

gate tests conducted in this study. The analyses included model selections for toughness and abrasion resistance, durability and soundness and overall performance (the worst rating) as shown in Table 47. For this part of the study, pavement performance was the dependent variable. The rated performance was assigned numerical values of 5, 3 and 1 for good, fair and poor performance, respectively.

Single-Variable Correlations

Results of single-variable correlations are summarized in Tables 50, 51, and 52 for toughness and abrasion resistance, durability and soundness and overall performance, respectively. Ten independent variables are incorporated in the correlations with pavement performance based on aggregate toughness and abrasion resistance. These include results from five toughness and abrasion resistance type tests as well as aggregate specific gravity and flat and elongated particles. Specific gravity is included because of the perception that denser aggregate are tougher and more abrasion resistant. Flat and elongated particle count is included because these types of particles are thought more susceptible to degradation during construction and in-service pavements.

Thirteen independent variables are incorporated in the correlations with pavement performance based on durability and soundness. These include results from three durability and soundness tests as well as specific gravity, absorption, dust-ratio from the LA abrasion test, methylene blue index on dust from the Los Angeles abrasion test, and the Micro-Deval abrasion loss. Specific gravity and absorption are included because of the perception that more absorptive (and logically less dense) aggregates are less durable. The relative quantity and methylene blue index on dust from the Los Angeles abrasion test, are included because more dust and more plastic dust are thought detrimental to pavement durability. The Micro-Deval test is included because water and agitation simulates environmental as well as mechanical degradation, (i.e., the Micro-Deval may be considered a durability and soundness as well as toughness and abrasion resistance test).

Thirteen independent variables were selected for incorporation in the correlations with overall pavement performance.

Rock Source	Description	Rock Source	Description	Rock Source	Description	Rock Source	Description
1	Traprock,NY	5	Gravel,NV	9	Limestone, IA	13	Sandstone,PA
2	Granite,GA	6	Limestone,IA	10	Gravel, PA	14	Limestone,MN
3	Steel Slag, IN	7	Granite,SC	11	Limerock,FL	15	Siltstone,VA
4	Gravel,MN	8	Gravel, MN	12	Limestone,TX	16	Basalt,OR

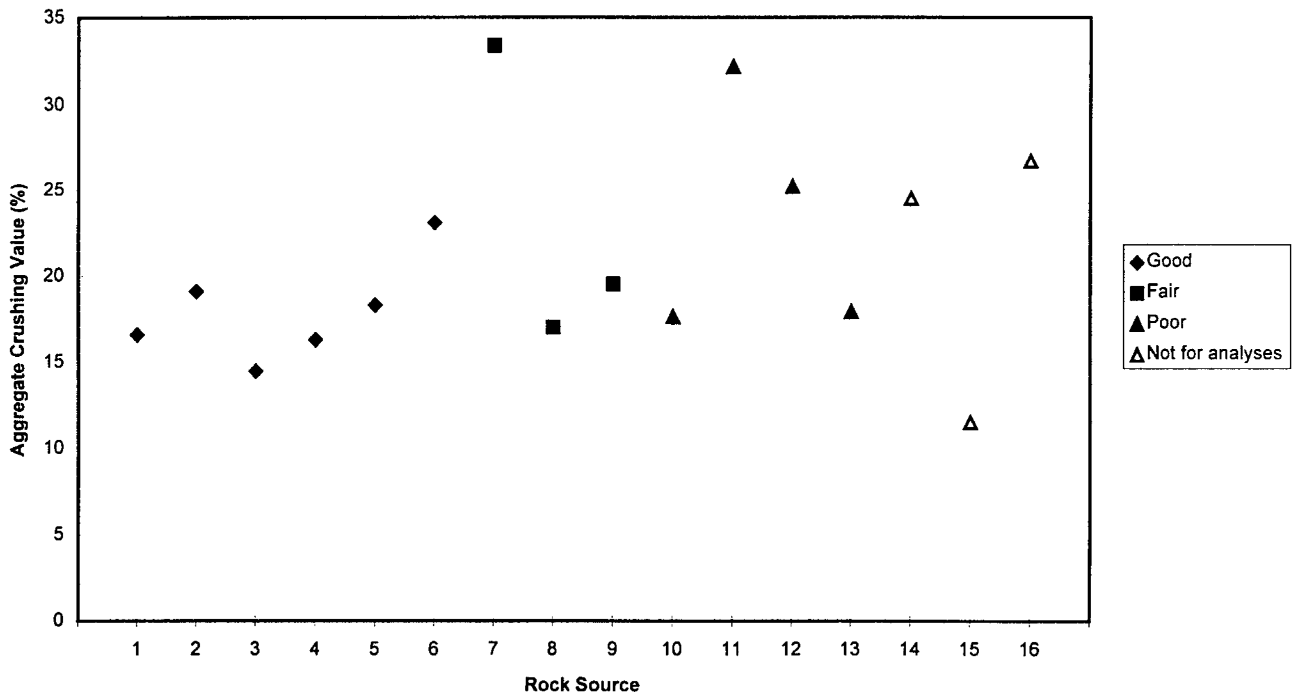


Figure 17. Pavement performance ratings and aggregate crushing value.

These include results from four toughness and abrasion resistance tests and three durability and soundness tests as well as specific gravity, absorption and methylene blue index. The selections were based on the strength of correlations with performance based on toughness and abrasion resistance and durability and soundness. Results from at least one of each general type of test were used. The strength of correlations between various tests were considered when eliminating or including a specific test.

The results in Table 50 indicate that Micro-Deval has the highest correlation coefficient R value ($R = -0.81$) and far exceeds the R values of the other tests. This correlation is also the only one with a significance level greater than 5 percent ($P \leq 0.0007$).

The results in Table 51 show several variables with relatively good correlations that are significant at a 5 percent level, but the two with the highest R and lowest P are Micro-Deval ($R = -0.87$, $P \leq 0.0001$) and magnesium sulfate soundness ($R = -0.81$, $P \leq 0.0004$).

Table 52 reveals several variables with relatively good correlations that are significant at 5 percent level but, again, the two with highest R and lowest P are Micro-Deval ($R = -0.85$, $P \leq 0.0001$) and magnesium sulfate soundness ($R = -0.79$, $P \leq 0.0004$).

Forward Selection Multiple Variables Procedure

The forward selection procedure was tried to see if multiple variable correlations could be found with improved correlation and significance. However, only the one variable correlations identified in Step One were found to be significant at a 5 percent confidence level. The independent variable selected for all three performances cases was the Micro-Deval loss. This was expected because Micro-Deval loss had the highest correlation coefficients and significance levels in the single variable analyses.

The single variable regression equations are as follows:

- Toughness and Abrasion Resistance
  Performance Rating = 6.053 − 0.167 (Micro-Deval Loss)
  $R^2 = 0.66$, $P \leq 0.0007$

Rock Source	Description	Rock Source	Description	Rock Source	Description	Rock Source	Description
1	Traprock,NY	5	Gravel,NV	9	Limestone, IA	13	Sandstone,PA
2	Granite,GA	6	Limestone,IA	10	Gravel, PA	14	Limestone,MN
3	Steel Slag, IN	7	Granite,SC	11	Limerock,FL	15	Siltstone,VA
4	Gravel,MN	8	Gravel, MN	12	Limestone,TX	16	Basalt,OR

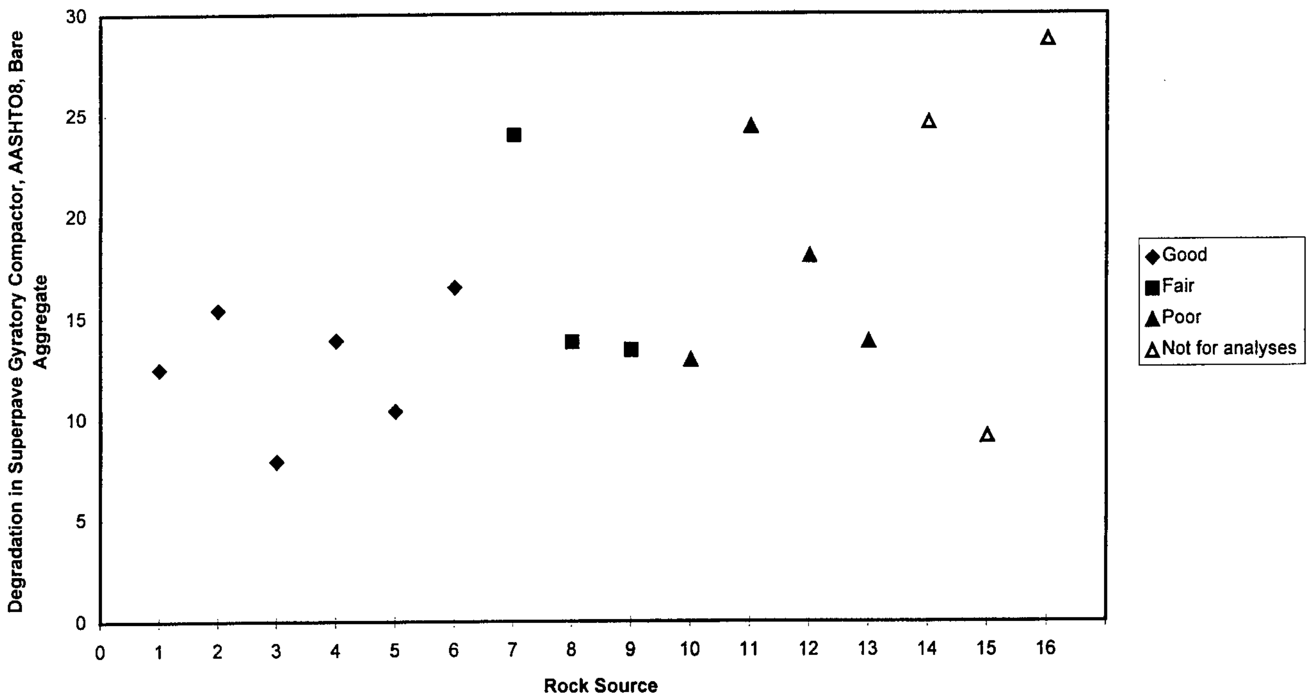


Figure 18. Pavement performance ratings and Superpave Gyratory Compactor, AASHTO 8, bare aggregate.

- Durability and Soundness
  Performance Rating = 6.473 − 0.166 (Micro-Deval Loss)
  $R^2 = 0.76, P \leq 0.0001$
- Overall
  Performance Rating = 5.940 − 0.158 (Micro-Deval Loss)
  $R^2 = 0.72, P \leq 0.0001$

Application of the above equations to source 4 (gravel, good performance), source 8 (gravel, fair performance), and source 13 (sandstone, poor performance) yields the following predictions:

		Predicted Performance		
Source	*Micro-Deval Loss, %*	*Toughness/ Abrasion*	*Durability/ Soundness*	*Overall*
4	9.6	4.4	4.9	4.4
8	14.6	3.6	4.0	3.6
13	34.0	0.4	0.8	0.6

The equations provide reasonable and similar predictions of performance for all three sources. The equation based on durability and soundness always suggests somewhat higher performance than the other two equations.

Summary

The qualitative visual examinations of plots of test values from various tests and pavement performance ratings, on the basis of toughness and abrasion resistance and durability and soundness, suggest Micro-Deval and magnesium sulfate loss are the two best indicators of potential pavement performance. Losses of 18 percent for both the Micro-Deval and magnesium sulfate soundness tests appear to separate good and fair aggregates from poor aggregates as illustrated in Figure 24. All aggregate sources, except source 15, Virginia siltstone, which was eliminated from the analyses, are properly grouped with this criterion. All poor and good plus fair performers are in the proper quadrants. No sources fall in the two quadrants where there would be conflicts between the tests.

Single variable correlations between performance ratings and tests indicate the Micro-Deval test has the highest and the magnesium sulfate test the second highest R values. For

Rock Source	Description	Rock Source	Description	Rock Source	Description	Rock Source	Description
1	Traprock,NY	5	Gravel,NV	9	Limestone,IA	13	Sandstone,PA
2	Granite,GA	6	Limestone,IA	10	Gravel,PA	14	Limestone,MN
3	Steel Slag, IN	7	Granite,SC	11	Limerock,FL	15	Siltstone,VA
4	Gravel,MN	8	Gravel,MN	12	Limestone,TX	16	Basalt,OR

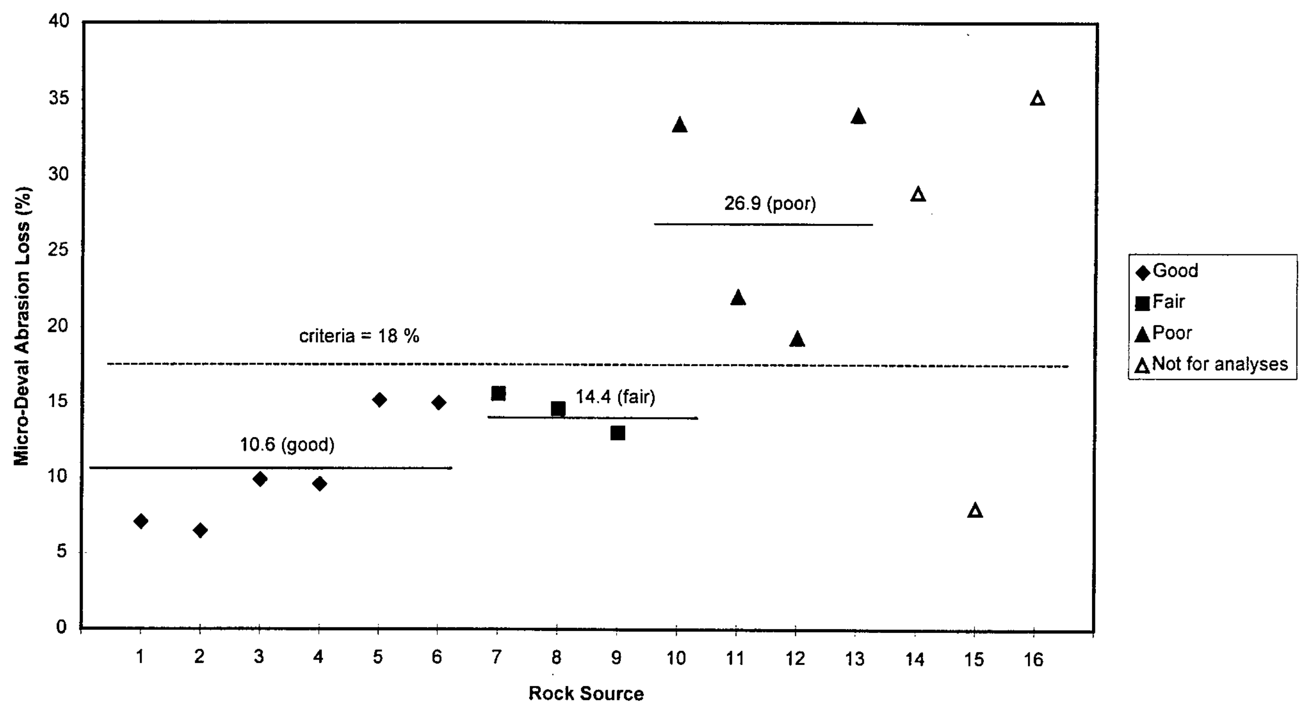


Figure 19. Pavement performance ratings and Micro-Deval abrasion loss (for toughness/abrasion resistance).

performance based on toughness and abrasion resistance only, the correlation with Micro-Deval loss is significant at a 5 percent level, but for durability and soundness and overall performance, correlations with both Micro-Deval and magnesium sulfate loss are significant at a 5 percent level.

The forward selection process provided only single-variable (Micro-Deval loss) equations with a 5 percent significance level for all three performance cases. No multiple variable equations were found.

Magnesium sulfate is recommended as a second test because of its identification as an important variable for performance based on durability and soundness, its history of use, its lack of required special equipment, and its somewhat lower variability.

Conclusion

The Micro-Deval test and magnesium sulfate soundness test (AASHTO T104) are best related to HMA performance in terms of popouts, raveling, and potholing. They are recommended for evaluating aggregate for HMA in pavements.

FINDINGS OF P200 MATERIAL CHARACTERIZATION STUDY

There are two performance parameters of interest in this section: permanent deformation (rutting), and fatigue cracking. If the material passing a 75 μm or No. 200 sieve (P200 material) acts as an extender of the asphalt cement binder, permanent deformation could occur because of an over-rich mix. Fatigue cracking can occur if the P200 material causes excessive stiffening of the asphalt binder and/or asphalt mixture. Moisture susceptibility of a mix may also be affected by the amount and nature of the P200 material, resulting in stripping which may, in turn, cause permanent deformation. The objective of this element of the study is to determine which aggregate tests used for characterizing the P200 materials are related to performance of HMA in terms of permanent deformation and fatigue cracking.

Characterization of P200 Materials Used

Six aggregate sources were chosen to represent a wide range of mineralogical composition and particle sizes.

Rock Source	Description	Rock Source	Description	Rock Source	Description	Rock Source	Description
1	Traprock,NY	5	Gravel,NV	9	Limestone,IA	13	Sandstone,PA
2	Granite,GA	6	Limestone,IA	10	Gravel,PA	14	Limestone,MN
3	Steel Slag, IN	7	Granite,SC	11	Limerock,FL	15	Siltstone,VA
4	Gravel,MN	8	Gravel,MN	12	Limestone,TX	16	Basalt,OR

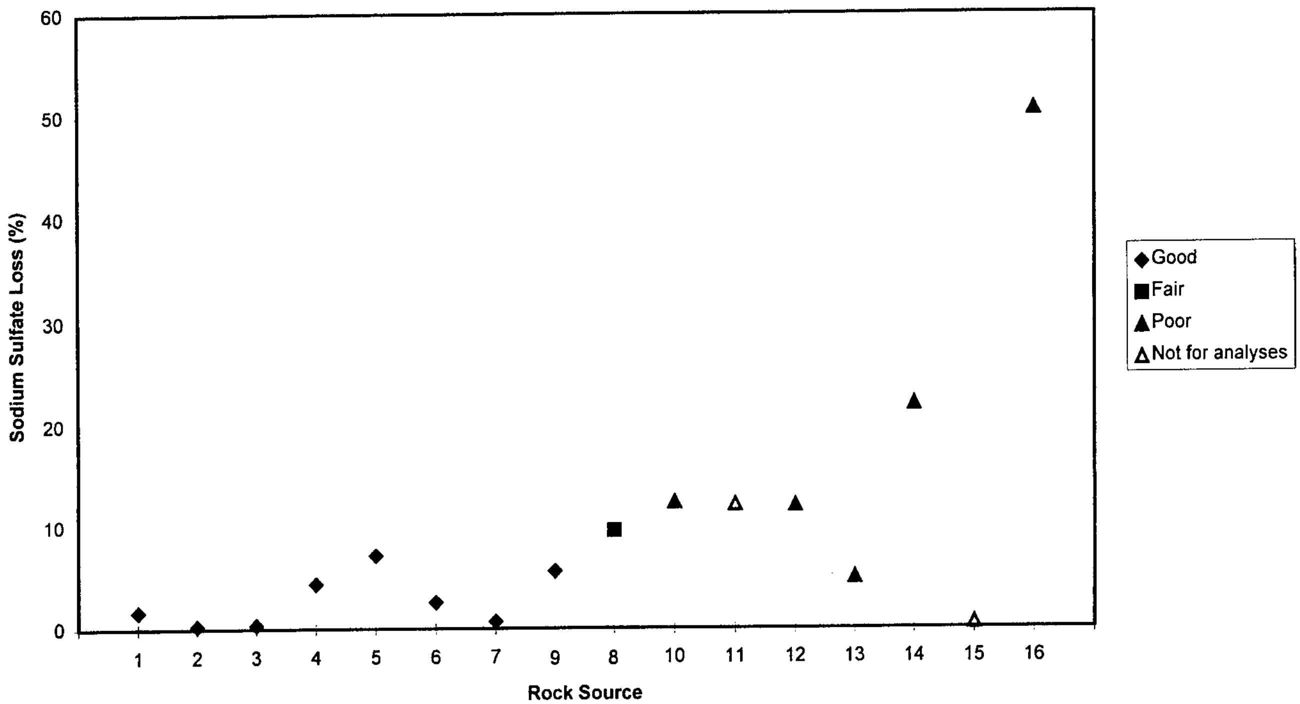


Figure 20. Pavement performance ratings and sodium sulfate soundness.

Table 53 contains a list and location of all P200 materials that were used. These materials were obtained by dry sieving fine aggregate parent rock over a 75 μm (No. 200) sieve.

The following tests were used to characterize the P200 materials.

- Rigden Voids (British Standard 812)
- Rigden Voids (Penn State Modified)
- Particle Size Analysis
- Methylene Blue Test - Ohio DOT Procedure
- Plasticity Index - AASHTO T 90
- German Filler Test - Koch Materials Company Procedure

The Rigden voids tests determine the void volume in a dry-compacted dust. The British Standard and Penn State Modified tests are based on the same concept but use different testing equipment and compactive effort.

Particle size analysis was conducted with a Coulter LF200 particle size analyzer. From this analysis, several parameters were determined. The relative fineness of an aggregate can be determined by the calculated fineness modulus (FM). The FM of the P200 material was calculated by dividing by 100 the sum of the percentages of P200 material coarser than 75, 50, 30, 20, 10, 5, 3, and 1 microns. The finer the aggregate, the smaller the FM. Parameters D10, D30, and D60 were also determined from particle size analysis. These parameters are the particle sizes that correspond to 10, 30, and 60 percent of the material passing. The specific surface area (SA) (cm^2/ml) was the final parameter obtained from this analysis.

The methylene blue test determines the nature and amount of potentially harmful clay and organic material that may be present in an aggregate.

As described earlier, the German filler test is a measure of the amount of mineral filler required to absorb 15 gm of hydraulic oil.

Table 54 contains the results of the P200 characterization tests. Each value is the average of three replicates.

Table 55 contains the correlation matrix between aggregate properties. The correlation coefficients (R values) are the first number in each cell. The second number in each cell is the statistical significance level (P) corresponding to the correlation coefficient.

Rigden voids (British Standard) and Rigden voids (Penn State Modified) have a good correlation ($R=0.78$, $P=0.06$)

Rock Source	Description	Rock Source	Description	Rock Source	Description	Rock Source	Description
1	Traprock,NY	5	Gravel,NV	9	Limestone,IA	13	Sandstone,PA
2	Granite,GA	6	Limestone,IA	10	Gravel,PA	14	Limestone,MN
3	Steel Slag, IN	7	Granite,SC	11	Limerock,FL	15	Siltstone,VA
4	Gravel,MN	8	Gravel,MN	12	Limestone,TX	16	Basalt,OR

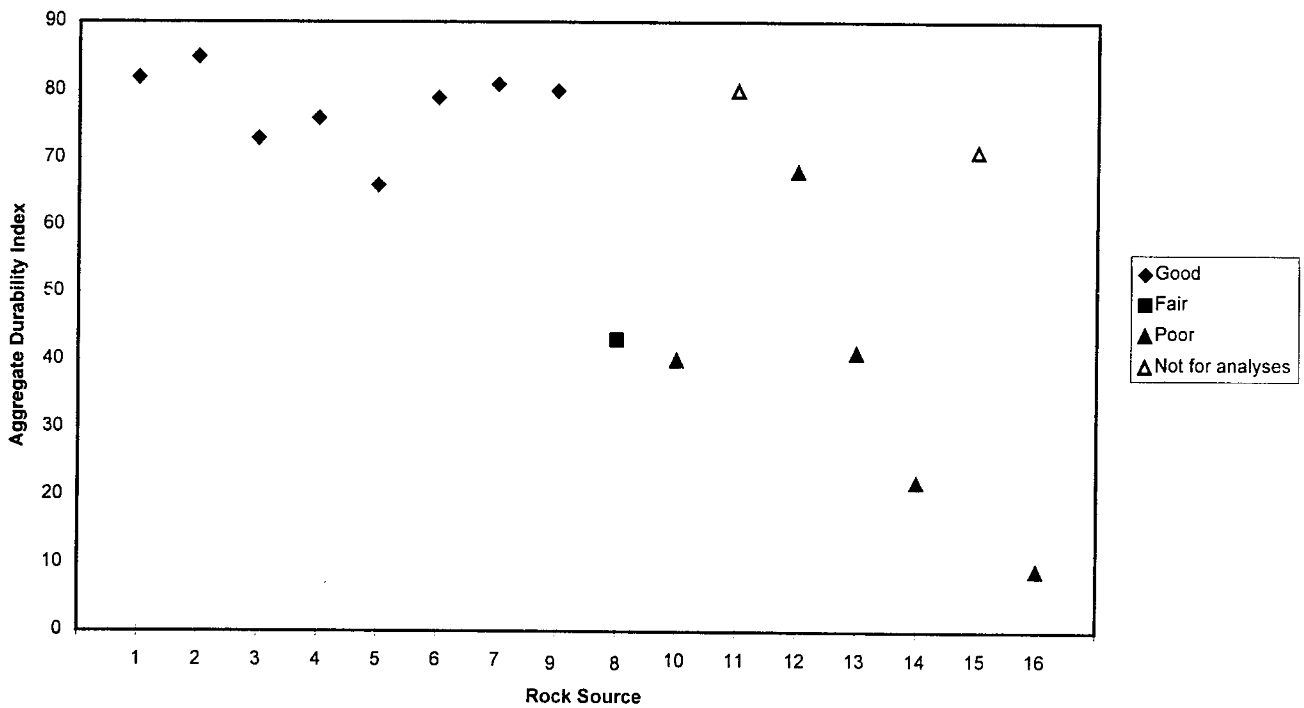


Figure 21. Pavement performance ratings and aggregate durability index.

with each other because both measure the voids in the compacted P200 material, although with different compactive efforts. Rigden voids (Penn State Modified) has an excellent correlation with the German filler test, whereas Rigden voids (British Standard) has only a fair correlation with the German filler test as follows:

	Correlation Coefficient R	*Significance Level, P*
Rigden voids (Penn State) versus German Filler	−0.95	0.004
Rigden voids (British Standard) versus German Filler	−0.75	0.086

The German filler test is based indirectly on the Rigden voids concept. If the Rigden voids are high, the amount of P200 material needed to reach the end point of the test is relatively low because more hydraulic oil is fixed by the high voids. The German filler test does not require any special equipment, is very simple to perform, and can be substituted for Rigden voids (Penn State Modified).

The test parameters FM, specific SA, D10, D30, and D60 are strongly related to each other as shown in Table 56. All correlations are significant at the 5 percent level.

Rigden voids (Penn State Modified) and Rigden voids (British Standard) do not have any correlation with the particle size parameters.

Characterization of Filler-Asphalt Mortars

When asphalt binder is mixed with aggregate, the P200 material mixes with the asphalt binder to form a fines/asphalt mortar. This mortar is assumed to have properties that differ from the original asphalt binder. The addition of fines to the asphalt binder can have two main effects. The fines can extend the asphalt and/or stiffen the binder. This modification could affect the mix properties. Therefore, it is necessary to determine first how each of the P200 materials modify the asphalt binders.

To determine the effect of each of the P200 aggregates on the mortar, two fines/asphalt (F/A) ratios were used for each aggregate. The two F/A ratios chosen were 0.8 and 1.5 by

Rock Source	Description	Rock Source	Description	Rock Source	Description	Rock Source	Description
1	Traprock,NY	5	Gravel,NV	9	Limestone,IA	13	Sandstone,PA
2	Granite,GA	6	Limestone,IA	10	Gravel,PA	14	Limestone,MN
3	Steel Slag, IN	7	Granite,SC	11	Limerock,FL	15	Siltstone,VA
4	Gravel,MN	8	Gravel,MN	12	Limestone,TX	16	Basalt,OR

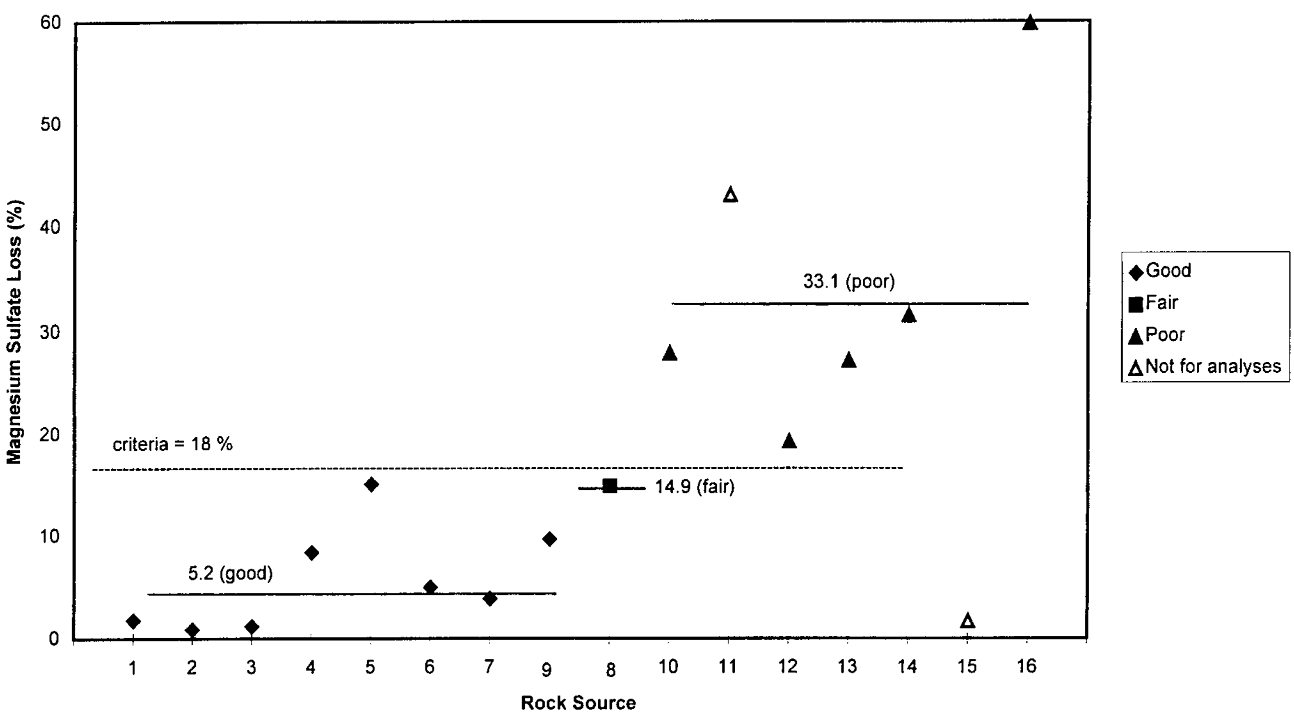


Figure 22. Pavement performance ratings and magnesium sulfate soundess.

weight. The 0.8 ratio represents a typical amount of P200 material in a dense-graded HMA mix (approximately 5 percent) by weight of HMA. The 1.5 ratio represents dense-graded mixes that contain excessive amounts of P200 material which are closer to the upper tolerance for P200 amount during production (approximately 8 percent by weight of HMA). Many states allow a F/A ratio range of 0.6 to 1.2. A Superpave PG 64-22 asphalt cement was used to make F/A mortar and subsequently the HMA mixtures.

The fines/asphalt mortars were tested at three different stages of aging. The Superpave dynamic shear rheometer was used to determine the following:

- G*/sinδ (rutting factor) of the unaged F/A mortar at 64°C,
- G*/sinδ (rutting factor) of the rolling thin film oven (RTFO) residue of the F/A mortar at 64°C, and
- G*sinδ (fatigue factor) of the pressure aging vessel (PAV) residue in the F/A mortar at 25°C.

A high value for G*/sinδ should indicate a greater resistance to rutting, while a high value for G*sinδ should indicate increased fatigue cracking. Table 57 shows the results of the tests on F/A mortars.

Table 58 contains the correlation matrix between mortar properties. All F/A mortar tests have very high correlation with each other (all have $P<0.05$) with the exception of the correlations made with the fatigue cracking parameter (G*sinδ) at F/A=0.8.

Table 59 shows the correlation between 0.8 F/A mortar properties and P200 aggregate properties. The first number in each cell is correlation coefficient R, the second number is the significance level P value.

Table 60 shows the best relationships from Table 59. The same independent variables appear for both unaged and RTFO residues. However, the correlations for G*sinδ are not significant. It is likely that more testing error is involved when the F/A system is aged in the PAV and also tested at intermediate temperature (25°C) for G*sinδ. There also could be different interactions between the asphalt binder and the different P200 materials, which could produce different aging results.

Table 61 shows the correlation between the 1.5 F/A mortar and the P200 properties.

Rock Source	Description	Rock Source	Description	Rock Source	Description	Rock Source	Description
1	Traprock,NY	5	Gravel,NV	9	Limestone,IA	13	Sandstone,PA
2	Granite,GA	6	Limestone,IA	10	Gravel,PA	14	Limestone,MN
3	Steel Slag, IN	7	Granite,SC	11	Limerock,FL	15	Siltstone,VA
4	Gravel,MN	8	Gravel,MN	12	Limestone,TX	16	Basalt,OR

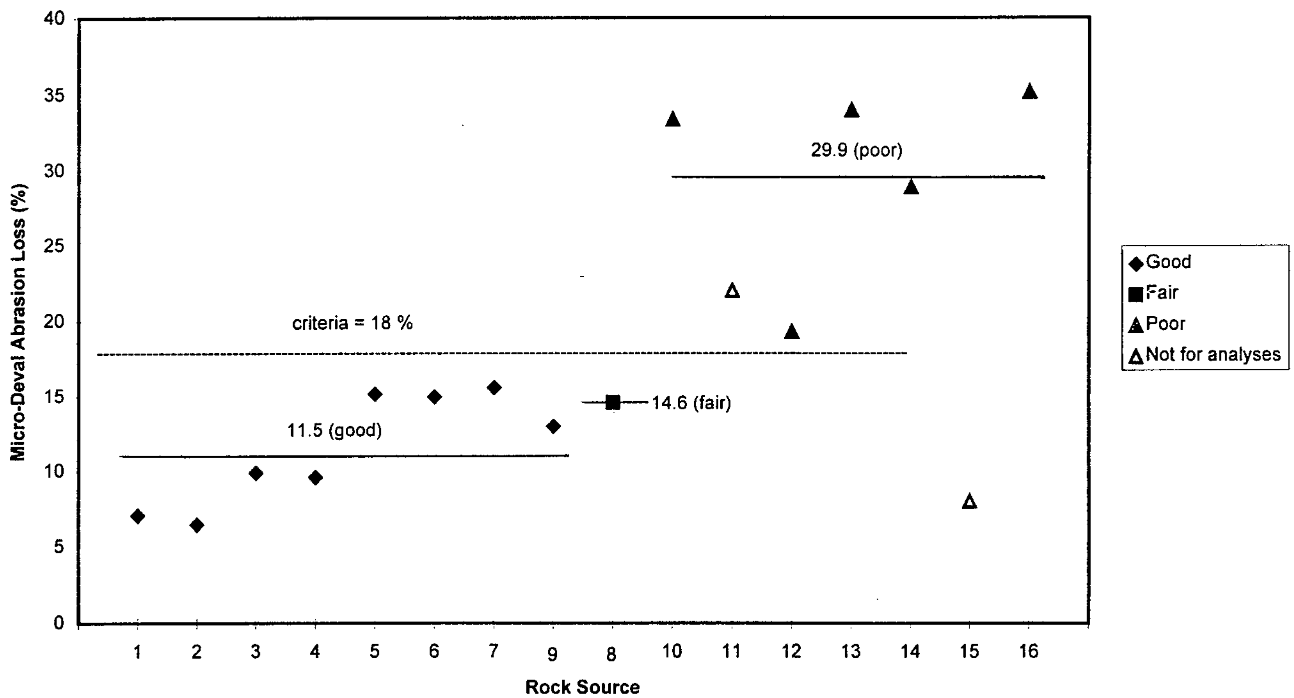


Figure 23. Pavement performance ratings and Micro-Deval abrasion loss (for durability and soundness).

TABLE 50 Summary of correlation between performance ratings and toughness/abrasion tests

Test	Correlation Coefficient R	Significance Level P
Superpave Gyratory Compactor, AASHTO 8 + Fine, Mix	-0.16	0.6110
Superpave Gyratory Compactor, AASHTO 8 + Fine, Bare Aggregate	-0.28	0.3466
Flat & Elongated Particles	-0.38	0.1966
Aggregate Impact Value	-0.41	0.1679
Aggregate Crushing Value	-0.41	0.1636
Superpave Gyratory Compactor, AASHTO 8, Mix	-0.44	0.1285
Superpave Gyratory Compactor, AASHTO 8, Bare Aggregate	-0.45	0.1266
Los Angeles Abrasion	-0.48	0.0955
Specific Gravity	0.52	0.0675
Micro-Deval	-0.81	0.0007

TABLE 51 Summary of correlation between performance ratings and durability/soundness tests

Test	Correlation Coefficient R	Significance Level P
Dust-Ratio from LA Abrasion	0.21	0.4644
Absorption	-0.38	0.1831
Methylene Blue Value	-0.38	0.1759
Specific Gravity	0.42	0.1397
AASHTO Freeze-Thaw, Pro. C	-0.58	0.0297
AASHTO Freeze-Thaw, Pro. B	-0.64	0.0145
Sodium Sulfate	-0.64	0.0129
Modified Canadian Freezing-Thawing	-0.67	0.0093
Canadian Freeze-Thaw	-0.68	0.0078
AASHTO Freeze-Thaw, Pro. A	-0.73	0.0033
Aggregate Durability Index	0.74	0.0024
Magnesium Sulfate	-0.81	0.0004
Micro-Deval Abrasion	-0.87	0.0001

TABLE 52 Summary of correlation between overall performance ratings and overall tests

Test	Correlation Coefficient R	Significance Level P
Methylene Blue Value	-0.20	0.4840
Absorption	-0.44	0.1001
Aggregate Impact Value	-0.45	0.0917
Los Angeles Abrasion	-0.48	0.0673
AASHTO Freeze-Thaw, Pro. B	-0.50	0.0566
Canadian Freeze-Thaw	-0.54	0.0380
Specific Gravity	0.55	0.0325
AASHTO Freeze-Thaw, Pro. C	-0.58	0.0221
Superpave Gyratory Compactor, AASHTO 8, Bare Aggregate	-0.59	0.0214
Aggregate Durability Index	0.63	0.0121
AASHTO Freeze-Thaw, Pro. A	-0.71	0.0032
Magnesium Sulfate	-0.79	0.0004
Micro-Deval	-0.85	0.0001

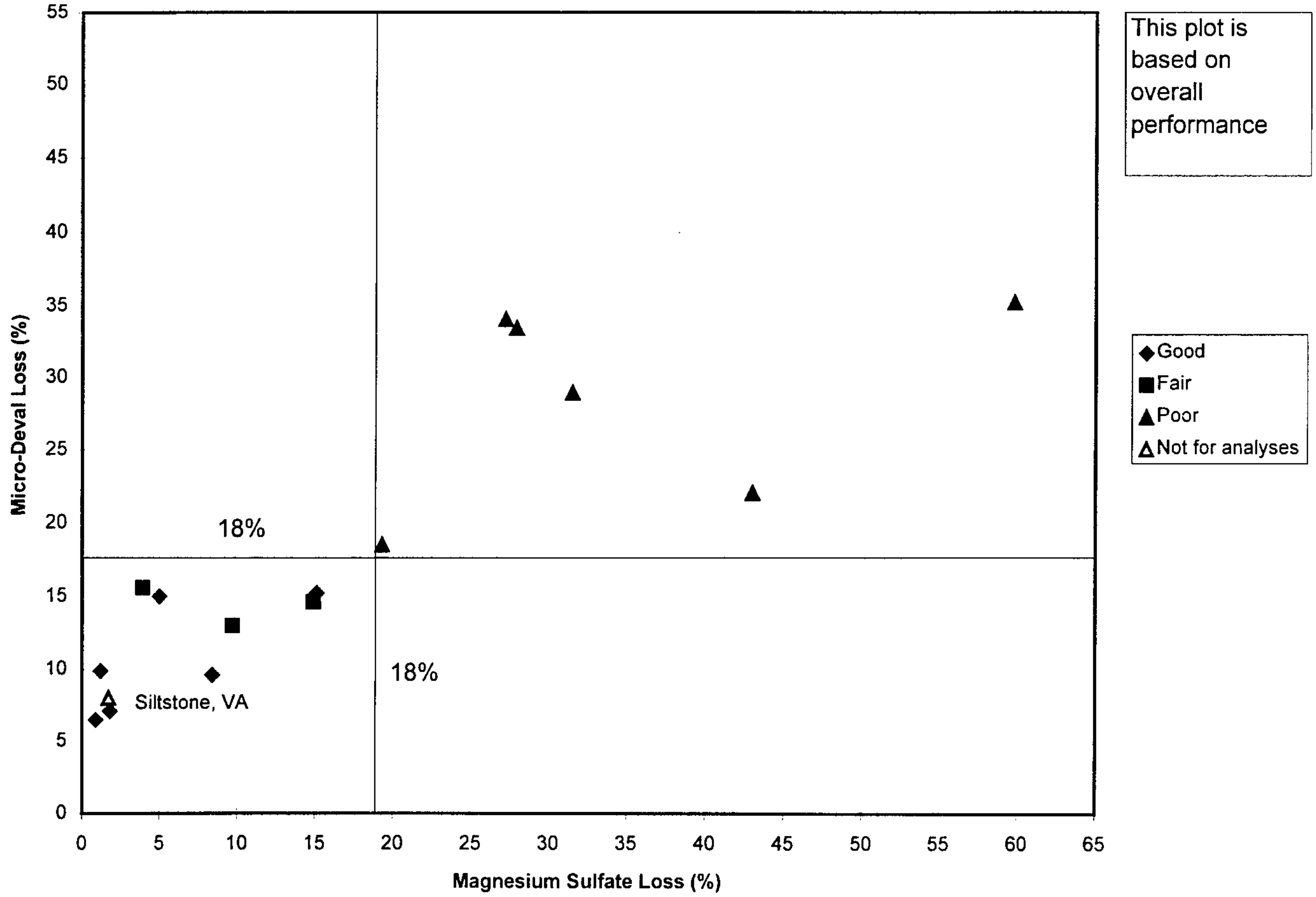


Figure 24. Magnesium sulfate soundness and Micro-Deval loss (%) criteria for eliminating poor performing aggregates.

Table 62 shows the best relationships from Table 61.

Methylene blue and German filler appear as significant variables affecting the G*/sinδ (rutting factor) and G*sinδ (fatigue factor) of the 1.5 F/A mortars. The size and gradation properties of the P200 material (D10, D30, D60, fineness modulus, and specific surface area) are not significantly related to either G*/sinδ or G*sinδ.

Forward Selection Multiple Regression Procedure

To determine if regression equations with multiple P200 properties would improve prediction of F/A mortar properties, the forward selection procedure in the SAS computer program was used. The results of this analysis are contained in Table 63.

In this forward selection procedure, the dependent variables are the rutting parameter, G*/sinδ (at high temperature), and the fatigue parameter, G*sinδ (at intermediate temperature), of the F/A system (mortar) and not of the HMA. The F/A binder system may act differently in a dense-graded HMA when combined with coarse aggregate and fine aggregate. This is important to keep in mind when reviewing the results of this statistical analysis. Table 63 gives the one- and two-variable models for all three dependent variables for both 0.8 and 1.5 F/A ratios.

TABLE 53 P200 description and location

Sample Number	Description	Location
P200-1	Pit Run Natural Sand	Alabama
P200-2	High Calcium Limestone	Alabama
P200-3	Dolomite	Alabama
P200-4	Granite	Georgia
P200-5	Blast Furnace Slag	Alabama
P200-6	Limerock	Florida

TABLE 54 P200 aggregate test results

Mix #	P200 -1	P200-2	P200-3	P200-4	P200-5	P200-6
Test	Natural Sand	Limestone	Dolomite	Granite	Blast Furnace Slag	Limerock
Specific Gravity	2.558	2.760	2.955	2.872	3.043	2.798
Rigden Void (British Standard), %	39.3	35.4	32.3	41.4	40.6	34.3
Rigden Void (Penn State Modified), %	53.8	38.0	38.9	45.5	49.8	38.5
Fineness Modulus	3.67	2.46	4.99	4.50	4.47	2.81
D10 (micron)	1.54	1.26	4.18	3.30	2.41	1.38
D30 (micron)	6.45	3.27	22.58	15.64	12.67	4.23
D60 (micron)	26.92	9.98	51.95	40.89	43.41	14.60
Specific Surface Area (cm^2/ml)	12900	17968	6207	7206	8752	15603
Methylene Blue	18.7	1.3	0.3	2.1	2.0	9.5
Plasticity Index	29	NP	NP	NP	NP	NP
German Filler Test	35	70	80	60	55	75

TABLE 55 Correlation matrix of P200 aggregate properties[a]

	Rigden Void, BS	Rigden Void, PS	FM	D10	D30	D60	SA	MB	German Filler
Rigden Void, BS	1.0	0.784 0.065	0.235 0.655	-0.073 0.890	-0 051 0.923	0.186 0.724	-0.250 0.633	0.166 0.753	-0.749 0.086
Rigden Void, PS		1.0	0.326 0.529	-0.092 0.863	-0.013 0.980	0.277 0.595	-0.268 0.607	0.557 0.251	-0.949 0.004
FM			1.0	0.899 0.015	0.936 0.006	0.996 0.0001	-0.990 0.0001	-0.302 0.561	-0.058 0.913
D10				1.0	0.992 0.0001	0.907 0.013	-0.919 0.010	-0.538 0.270	0.321 0.535
D30					1.0	0.947 0.004	-0.942 0.005	-0.503 0.309	0.261 0.617
D60						1.0	-0.983 0.0004	-0.360 0.483	-0.001 0.998
SA							1.0	0.347 0.500	0.000 0.999
MB								1.0	-0.680 0.137
German Filler									1.0

[a] Top values are correlation coefficients R and bottom values are significance levels P in each cell.

TABLE 56 Relationship between parameters found by the particle size analyzer

Parameters	R	P
FM vs. D60	0.996	0.0001
FM vs. SA	-0.99	0.0001
FM vs. D30	0.94	0.006
FM vs. D10	0.90	0.014
SA vs. D60	-0.98	0.0004
SA vs. D30	-0.94	0.005
SA vs. D10	-0.92	0.009
D60 vs. D30	0.95	0.004
D60 vs. D10	0.91	0.012

Rutting. High values of G*/sinδ at high temperatures indicate that the F/A mortars should provide increased resistance to rutting. Four 2-variable models involving G*/sinδ of unaged F/A mortars and RTFO residue of F/A mortars at two F/A levels of 0.8 and 1.5 are given in Table 63.

Two of the models have coefficient of determination or R^2 values of 0.99 or more and significance levels or P values of 0.003 to 0.0006. On the basis of these two best models for G*/sinδ (rutting factor), methylene blue is considered to be the primary independent variable and German filler is considered to be the secondary independent variable. The other two models have German filler as the primary independent variable with Rigden voids, British Standard as the secondary variable.

Fatigue. High values of G*sinδ (fatigue factor) of F/A mortar should mean decreased resistance to fatigue cracking. The two 2-variable models for G*sinδ for 0.8 and 1.5 F/A ratio show methylene blue as the primary independent variable. German filler is the secondary independent variable in the 1.5 F/A mortar model which has an R^2 value of 0.86 and is significant at a 0.05 level. D30 is the secondary variable in the 0.8 F/A mortar model which is not significant at a 0.05 level.

Methylene blue and German filler are considered to be the tests which best predict potential rutting and fatigue cracking contribution of the F/A mortar to the HMA.

Mixture Validation Tests

There were 12 mixes evaluated in this section. The six P200 aggregates in both 0.8 and 1.5 F/A ratios were combined with limestone coarse and fine aggregate to produce

TABLE 57 F/A mortar test results

Aggregate	Fines/Asphalt Ratio (by weight)	G*/sinδ Unaged Mortar at 64°C (kPa)	G*/sinδ RTFO at 64°C (kPa)	G*sinδ PAV at 25°C (kPa)
P200-1A	0.8	5.52	12.41	13915
P200-2A	0.8	2.83	5.79	7101
P200-3A	0.8	3.04	4.78	10390
P200-4A	0.8	3.19	6.95	6639
P200-5A	0.8	3.41	6.91	8047
P200-6A	0.8	3.02	6.84	7149
P200-1B	1.5	10.91	28.06	24702
P200-2B	1.5	5.90	14.00	15281
P200-3B	1.5	5.35	10.11	11303
P200-4B	1.5	6.45	14.15	11278
P200-5B	1.5	6.80	14.08	13824
P200-6B	1.5	6.68	12.84	14139

TABLE 58 Correlation between tests on F/A mortars

	G*/sinδ (Unaged) 0.8	G*/sinδ (RTFO) 0.8	G*sinδ (PAV) 0.8	G*/sinδ (Unaged) 1.5	G*/sinδ (RTFO) 1.5	G*sinδ (PAV) 1.5
G*/sinδ (Unaged) 0.8	1.0	0.962 0.002	0.866 0.026	0.975 0.0009	0.964 0.002	0.910 0.012
G*/sinδ (RTFO) 0.8		1.0	0.710 0.114	0.996 0.0001	0.980 0.0006	0.918 0.010
G*sinδ (PAV) 0.8			1.0	0.757 0.082	0.749 0.086	0.766 0.076
G*/sinδ (Unaged) 1.5				1.0	0.981 0.0005	0.938 0.006
G*/sinδ (RTFO) 1.5					1.0	0.955 0.003
G*sinδ (PAV) 1.5						1.0

[a] Top values are correlation coefficients R and bottom values are significance levels P in each cell.

validation mixes. All limestone was washed over a 75 μm (No. 200) sieve prior to batching to remove the P200 material. Limestone was chosen as the coarse and fine aggregate so that moisture susceptibility would not be caused by the base aggregate. Moisture susceptibility differences, if any, can then be attributed to the effect of the P200 material. Figures 6 shows the HMA mix gradation used for the 0.8 F/A ratio (5 percent passing 75 μm sieve). The HMA mix gradation for the 1.5 F/A ratio was same as that for 0.8 F/A ratio except it had 8 percent passing 75 μm sieve.

A Superpave PG 64-22 grade asphalt cement was used in all HMA mix testing. Optimum asphalt content was deter-

TABLE 59 Correlation matrix between 0.8 F/A mortar and P200 aggregate properties[a]

P200 Test Property	G*/sinδ (Unaged)	G*/sinδ (RTFO)	G*sinδ (PAV)
Rigden Voids (British Standard)	0.395 0.439	0.490 0.324	-0.021 0.969
Rigden Voids (Penn State Method)	0.826 0.043	0.809 0.05	0.572 0.235
Fineness Modulus	0.053 0.920	-0.126 0.812	0.218 0.678
D10	-0.267 0.609	-0.446 0.375	0.007 0.989
D30	-0.204 0.698	-0.400 0.433	0.078 0.884
D60	-0.004 0.994	-0.191 0.717	0.188 0.722
Specific Surface Area	0.025 0.963	0.180 0.733	-0.114 0.830
Methylene Blue	0.865 0.026	0.917 0.010	0.690 0.129
German Filler	-0.888 0.018	-0.912 0.011	-0.588 0.220

[a] Top values are correlation coefficients R and bottom values are significance levels P in each cell.

TABLE 60 Best relationships for 0.8 F/A mortars and P200 properties

F/A Mortar	Independent Variable	Dependent Variable	R	P
Unaged	German Filler	G*/sinδ	-0.89	0.02
	Methylene Blue	G*/sinδ	0.86	0.03
	Rigden Voids (Penn State Method)	G*/sinδ	0.83	0.04
RTFO	Methylene Blue	G*/sinδ	0.92	0.01
	German Filler	G*/sinδ	-0.91	0.01
	Rigden Voids (Penn State Method)	G*/sinδ	0.81	0.05
PAV	Methylene Blue	G*sinδ	0.69	0.13*
	German Filler	G*sinδ	-0.59	0.22*
	Rigden Voids (Penn State Method)	G*sinδ	0.57	0.23*

* Not significant at 0.05 level.

mined by Superpave volumetric mix design for a mix containing all limestone aggregate (including the P200 fraction) using the 0.8 F/A ratio. Two specimens (150-mm diameter by 50-mm height) were compacted in the Superpave gyratory compactor at four asphalt contents. An asphalt content of 5.3 percent gave 4 percent air voids at N_{design} (119 gyrations, for intermediate design traffic level of 10^7 ESALs). This asphalt content was used for all validation mixes. Specimens were made to be tested by the SST (frequency sweep at constant height and simple shear at constant height) and the IDT. These samples were aged for 4 hr, then heated to compaction temperature and compacted. The testing of the compacted specimens was performed by the Asphalt Institute. Table 64 contains the VMA and VTM for all 12 mixtures. The values shown are averages of two samples. A mix designation with "A" suffix indicates a 0.8 F/A ratio while a "B" suffix represents a 1.5 F/A ratio.

Table 65 shows the average (two samples) VTM and VMA for the slab specimens prepared for testing in the Hamburg wheel tracking device.

The following three test parameters, which are used in the Superpave intermediate mix analysis, were used to determine the propensity of the HMA mixtures to permanent deforma-

TABLE 61 Correlation matrix between 1.5 F/A mortar and P200 aggregate properties[a]

P200 Test Property	G*/sinδ (Unaged)	G*/sinδ (RTFO)	G*sinδ (PAV)
Rigden Voids (British Standard)	0.435 0.389	0.443 0.379	0.219 0.676
Rigden Voids (Penn State Method)	0.801 0.06	0.775 0.070	0.629 0.181
Fineness Modulus	-0.120 0.821	-0.165 0.755	-0.340 0.509
D10	-0.443 0.379	-0.458 0.361	-0.600 0.209
D30	-0.389 0.446	-0.413 0.416	-0.546 0.263
D60	-0.181 0.732	-0.225 0.668	-0.383 0.454
Specific Surface Area	0.183 0.728	0.232 0.658	0.425 0.400
Methylene Blue	0.927 0.008	0.875 0.023	0.891 0.017
German Filler	-0.899 0.015	-0.909 0.012	-0.791 0.061

[a] Top values are correlation coefficients R and bottom values are significance levels P in each cell.

TABLE 62 Best relationships between 1.5 F/A mortar and P200 properties

F/A System	Independent Variable	Dependent Variable	R	P
Unaged	Methylene Blue	G*/sinδ	0.93	0.008
	German Filler	G*/sinδ	-0.90	0.01
	Rigden Voids (Penn State Method)	G*/sinδ	0.80	0.055
RTFO	German Filler	G*/sinδ	-0.91	0.01
	Methylene Blue	G*/sinδ	0.88	0.02
	Rigden Voids (Penn State Method)	G*/sinδ	0.78	0.07*
PAV	Methylene Blue	G*sinδ	0.89	0.02
	German Filler	G*sinδ	-0.79	0.06*
	Rigden Voids (Penn State Method)	G*sinδ	0.63	0.18*

* Not significant at the 0.05 level.

TABLE 63 P200 regression equations

0.8 F/A Mortar						
Performance Parameter	Step	Dependent	Independent	Equation	R^2	P
Rutting	1	G*/sinδ (Unaged)	German Filler	G*/sinδ=6.928-0.0548(German Filler)	0.79	0.018
Rutting	2	G*/sinδ (Unaged)	Rigden Voids (British Standard)	G*/sinδ=14.932-0.167(Rigden voids,British Standard) - 0.083(German Filler)	0.96	0.009
Rutting	1	G*/sinδ (RTFO)	Methylene Blue	G*/sinδ=5.362+0.3395(Methylene Blue)	0.84	0.01
Rutting	2	G*/sinδ (RTFO)	German Filler	G*/sinδ=11.56+0.205(Methylene Blue)-0.087(German Filler)	0.996	0.0003
Fatigue	1	G*sinδ (PAV)	Methylene Blue	G*Xsinδ=7344.72+270.45(Methylene Blue)	0.48	0.13
Fatigue	2	G*sinδ (PAV)	D30	G*Xsinδ=4417.4+212.15(D30)+382.73(Methylene Blue)	0.72	0.15
1.5 F/A Mortar						
Performance Parameter	Step	Dependent	Independent	Equation	R^2	P
Rutting	1	G*/sinδ (Unaged)	Methylene Blue	G*/sinδ=5.567+0.256(Methylene Blue)	0.859	0.008
Rutting	2	G*/sinδ (Unaged)	German Filler	G*/sinδ=9.882+0.162(Methylene Blue)-0.0605(German Filler)	0.993	0.0006
Rutting	1	G*/sinδ (RTFO)	German Filler	G*/sinδ=37.51-0.352(German Filler)	0.827	0.012
Rutting	2	G*/sinδ (RTFO)	Rigden Voids (British Standard)	G*/sinδ=81.598-0.921(Rigden Voids British Standard) - 0.509(German Filler)	0.96	0.009
Fatigue	1	G*sinδ (PAV)	Methylene Blue	G*sinδ=11593.73+618.12(Methylene Blue)	0.79	0.02
Fatigue	2	G*sinδ (PAV)	German Filler	G*sinδ=19073.7+455.32(Methylene Blue) - 104.96 (German Filler)	0.86	0.05

TABLE 64 Average VTM and VMA for Superpave mix validation test specimens (P200 study)

Mix Designation	F/A Ratio	VTM	VMA
P200-1A	0.8	3.3	13.4
P200-2A	0.8	2.5	13.0
P200-3A	0.8	3.4	14.0
P200-4A	0.8	3.3	13.8
P200-5A	0.8	3.4	14.2
P200-6A	0.8	2.7	13.2
P200-1B	1.5	2.3	12.3
P200-2B	1.5	1.8	12.4
P200-3B	1.5	2.9	13.9
P200-4B	1.5	3.0	13.7
P200-5B	1.5	3.7	14.8
P200-6B	1.5	1.8	12.6

tion (rutting) and fatigue cracking. These test parameters were already discussed under coarse aggregate particle shape and texture.

1. $G^*/\sin\delta$ at 0.1 Hz
2. *Slope (m)* of the frequency versus *G* plot*
3. $G^*\sin\delta$ at 1.0 Hz

Moisture Susceptibility (Stripping)

The Hamburg wheel tracking test was used to evaluate moisture susceptibility. The point (number of cycles) where the slope of the rutting line and the slope of the stripping line intersect is called the inflection point. This is the point where stripping is assumed to have been initiated. Inflection point is the parameter of interest from this test. AASHTO T 283 was also used to measure the moisture-susceptibility of the HMA mixes in terms of tensile strength ratio (TSR). Table 66 contains all mixture validation test results.

Although correlations were made earlier between the P200 aggregate properties and the F/A mortar properties, the main objective of this section is to correlate the P200 aggregate properties with HMA properties determined by the mix validation tests. Table 67 shows the correlation matrix between

TABLE 65 Average VTM and VMA for Hamburg wheel tracking slab specimens (P200 study)

Mix Designation	F/A Ratio	VTM	VMA
P200-1A	0.8	7.4	17.0
P200-2A	0.8	7.2	17.2
P200-3A	0.8	7.9	18.0
P200-4A	0.8	7.8	17.9
P200-5A	0.8	7.7	18.0
P200-6A	0.8	7.2	17.2
P200-1B	1.5	6.5	16.1
P200-2B	1.5	6.6	16.8
P200-3B	1.5	6.9	17.4
P200-4B	1.5	6.6	17.0
P200-5B	1.5	6.7	17.4
P200-6B	1.5	5.8	16.1

TABLE 66 P200 mixture validation test results

Mix Designation	Rutting Parameters: High Temperature (40°C)		Fatigue Cracking Parameter: Intermediate Temperature (20°C)	Moisture Susceptibility Parameters	
	G*/sinδ @ 0.1 hz, psi	m	G*sinδ @ 1.0 hz psi	TSR Percent	Inflection Point (Hamburg)
1A	8722	0.43373	98051	66.2	7000
2A	9830	0.40572	89829	64.6	8400
3A	11034	0.40931	97967	51.7	20000
4A	9682	0.43596	88811	55.0	8600
5A	11271	0.40282	85644	57.5	10000
6A	12934	0.39183	93707	64.1	10000
1B	21700	0.31559	109560	64.7	6000
2B	21752	0.30167	98782	59.9	14400
3B	10269	0.40396	93318	52.4	9800
4B	11872	0.37874	84319	54.7	6400
5B	9510	0.41171	86723	64.5	8000
6B	25900	0.29158	95502	68.1	5000

TABLE 67 Correlation matrix between P200 aggregate properties and HMA properties (0.8 F/A)[a]

	Rutting		Fatigue	Stripping	
	G*/sinδ @ 0.1 hz	m	G*sinδ @ 1.0 hz	TSR	Inflection Point
Rigden Voids (British Standard)	-0.468 0.35	0.599 0.21	-0.556 0.25	0.033 0.95	-0.665 0.15
Rigden Voids (Penn State Method)	-0.526 0.28	0.599 0.21	-0.051 0.92	0.157 0.77	-0.463 0.36
Fineness Modulus	-0.094 0.86	0.365 0.48	0.039 0.94	-0.865 0.03	0.561 0.25
D10	0.007 0.99	0.248 0.63	0.110 0.84	-0.968 0.001	0.758 0.08
D30	0.01 0.99	0.228 0.66	0.113 0.83	-0.961 0.002	0.766 0.08
D60	-0.048 0.93	0.294 0.57	0.016 0.98	-0.886 0.02	0.606 0.20
Specific Surface Area	0.049 0.92	-0.367 0.47	0.009 0.99	0.896 0.02	-0.543 0.27
Methylene Blue	-0.255 0.63	0.306 0.56	0.534 0.28	0.693 0.13	-0.476 0.34
German Filler	0.657 0.16	-0.644 0.17	-0.003 0.99	-0.409 0.42	0.647 0.17

[a] Top values are correlation coefficients R and bottom values are significance levels P in each cell.

the P200 aggregate properties and the HMA properties at the 0.8 F/A gradation. Table 68 shows the correlation matrix between the P200 aggregate properties and HMA properties at the 1.5 F/A gradation. The first value in each cell is the correlation coefficient or R value. The second value in each cell is the significance level or P value.

G*/sinδ @ 0.1 Hz and m (the slope of the best fit line on the frequency versus G* plot) were the two HMA parameters chosen to indicate the rutting potential. Neither of these parameters correlates to any of the P200 aggregate tests at a significant level ($P<0.05$) in the 0.8 F/A ratio gradation. However, good correlations were obtained between the rutting parameters and the gradation indicators (fineness modulus, D10, D30, D60, and specific surface area) in the 1.5 F/A ratio gradation.

There were no significant correlations between P200 properties and G*sinδ (fatigue factor) in either 0.8 F/A ratio or 1.5 F/A ratio gradations. TSR correlated well with the gradation parameters at the 0.8 F/A ratio. However, with this exception, no significant correlations were seen between stripping parameters and P200 aggregate tests at either the 0.8 F/A ratio or the 1.5 F/A ratio.

The correlations are generally better for the F/A ratio of 1.5 (because higher amounts of P200 were used) than the F/A ratio of 0.8. Therefore, P200 tests can be better related to HMA performance at F/A ratio of 1.5, which will be primarily used in this project to select the P200 tests which are related to HMA performance. Table 69 shows some important relationships between P200 aggregate properties and HMA properties using the 1.5 F/A ratio (from Table 68). It appears that the fineness of P200 material expressed by the test parameters D60, D30, D10, fineness modulus, and specific surface area is significantly related to the permanent deformation of HMA at high concentration levels of P200 in the mix.

No significant relationships are observed between P200 aggregate properties and HMA rutting parameters (G*/sinδ @ 0.1 hz and m) or HMA fatigue parameter (G*sinδ) at an F/A ratio of 0.8. This indicates that at low concentration levels of P200, the effect on rutting and fatigue is not statistically significant.

Table 70 shows some of the significant relationships between the P200 aggregate tests and resistance to stripping using the mixture tests from the F/A ratio of 1.5.

TABLE 68 Correlation matrix between P200 aggregate properties and HMA properties (1.5 F/A)[a]

	Rutting		Fatigue	Stripping	
	G*/sinδ @ 0.1 hz	m	G*sinδ @ 1.0 hz	TSR	Inflection Point
Rigden Voids (British Standard)	-0.293 0.57	0.251 0.63	-0.257 0.62	0.143 0.79	-0.355 0.49
Rigden Voids (Penn State Method)	-0.201 0.70	0.221 0.67	0.206 0.70	0.286 0.58	-0.465 0.35
Fineness Modulus	-0.901 0.01	0.926 0.008	-0.488 0.33	-0.584 0.22	-0.249 0.63
D10	-0.838 0.04	0.836 0.04	-0.560 0.25	-0.824 0.04	-0.032 0.95
D30	-0.871 0.02	0.881 0.02	-0.532 0.28	-0.771 0.07	-0.049 0.92
D60	-0.920 0.009	0.948 0.004	-0.511 0.30	-0.590 0.22	-0.192 0.72
Specific Surface Area	0.895 0.016	-0.915 0.01	0.572 0.24	0.609 0.20	0.288 0.58
Methylene Blue	0.615 0.19	-0.559 0.25	0.776 0.07	0.613 0.20	-0.559 0.25
German Filler	-0.05 0.92	0.060 0.91	-0.413 0.42	-0.361 0.48	0.359 0.49

[a] Top values are correlation coefficients R and bottom values are significance levels P in each cell.

TABLE 69 Best relationships between P200 properties and HMA properties (1.5 F/A)[a]

Parameter	Independent Variable	Dependent Variable	Correlation Coefficient R	Significance Level P
Permanent Deformation	D60	G*/sinδ	-0.92	0.009
Permanent Deformation	Fineness Modulus	G*/sinδ	-0.90	0.014
Permanent Deformation	Specific Surface Area	G*/sinδ	0.89	0.016
Permanent Deformation	D30	G*/sinδ	-0.87	0.02
Permanent Deformation	D10	G*/sinδ	-0.84	0.04
Permanent Deformation	D60	m	0.95	0.004
Permanent Deformation	Fineness Modulus	m	0.93	0.008
Permanent Deformation	Specific Surface Area	m	0.92	0.01
Permanent Deformation	D30	m	0.88	0.002
Permanent Deformation	D10	m	0.84	0.04
Fatigue Cracking	Methylene Blue	G*sinδ	0.78	0.07 [a]
Fatigue Cracking	Specific Surface Area	G*sinδ	0.57	0.24 [a]
Fatigue Cracking	D10	G*sinδ	-0.56	0.25 [a]

[a] Not significant at the 0.5 level.

The fineness of the P200 material (especially D10) appears to affect the retained tensile strength (TSR) significantly. Because parent rocks (limestone) of the coarse aggregate and fine aggregate are the same in all mixes (only the P200 is different), the effect of binder stiffening (caused by the P200 material) appears to be dominant in these mixes. The smaller the size of P200 (especially D10), the more the binder is being modified and/or extended and thus gives increased resistance to stripping in the AASHTO T 283 test.

No P200 aggregate tests has any significant relationship with stripping when measured by the Hamburg wheel tracking device. Methylene blue has the highest (although insignificant at the 0.05 level) relationship with the inflection point. Obviously, the Hamburg wheel tracking test, which is conducted with HMA slabs submerged in hot water (50°C) and subjected to mechanical action, is significantly different than the stripping process in AASHTO T 283, which does not involve any mechanical action.

Table 71 shows some of the significant relationships between the P200 aggregate tests and resistance to stripping using the mixture tests with the F/A ratio of 0.8. The relationships are shown in decreasing order of significance.

Similar to the results obtained in mixes with the F/A ratio of 1.5, the fineness of the P200 material has a significant effect on the TSR.

As also observed in mixes with the F/A ratio of 1.5, no P200 test has any significant relationship with stripping when measured by the Hamburg wheel tracking device. Methylene blue is the only independent variable which has the highest (but not significant at the 0.05 level) correlation with inflection point ($R=-0.48$, $P=0.34$).

Forward Selection Procedure

Table 72 contains the aggregate tests selected in the forward selection procedure and the corresponding regression equations relating the P200 aggregate tests to the HMA performance parameters.

TABLE 70 Relationships between P200 tests and stripping for 1.5 F/A

Parameter	Independent Variable	Dependent Variable	Correlation Coefficient R	Significance Level P
Stripping	D10	TSR	-0.82	0.04
Stripping	D30	TSR	-0.77	0.07 [a]
Stripping	Methylene Blue	TSR	0.61	0.20 [a]
Stripping	Specific Surface Area	TSR	0.61	0.20 [a]
Stripping	D60	TSR	-0.59	0.22 [a]
Stripping	Methylene Blue	Inflection Point	-0.56	0.25 [a]
Stripping	Specific Surface Area	Inflection Point	0.29	0.58 [a]
Stripping	Fineness Modulus	Inflection Point	-0.25	0.63 [a]

[a] Not significant at the 0.05 level.

TABLE 71 Significant relationships between P200 tests and stripping for 0.8 F/A

Parameter	Independent Variable	Dependent Variable	Correlation Coefficient R	Significance Level P
Stripping	D10	TSR	-0.97	0.0015
Stripping	D30	TSR	-0.96	0.0023
Stripping	Specific Surface Area	TSR	-0.90	0.015
Stripping	D60	TSR	-0.89	0.02
Stripping	Fineness Modulus	TSR	-0.86	0.03
Stripping	Methylene Blue	TSR	0.69	0.12

The selection of the P200 aggregate tests that best relate to the HMA performance properties will be based solely on the information taken from the 1.5 F/A gradation testing section. This is because the 1.5 F/A ratio seems to correlate much better with the HMA performance properties than does the 0.8 F/A ratio because of an increased amount of P200 material in the mix. The models of the 0.8 F/A ratio generally have low coefficient of determination or R^2 values and insignificant P values.

Permanent Deformation

G/sinδ @ 0.1 Hz at high temperature.* High G*/sinδ values indicate increased resistance to permanent deformation or rutting. The two-variable model (see Table 72 and Figure 25) gives D60 as the primary independent variable and methylene blue as the second independent variable. The coefficient of determination or R^2 value of this model is 0.94 (P=0.015) which is excellent. As the particle size (at 60 percent passing) decreases, the G*/sinδ (stiffness or resistance to rutting) increases. It appears that the finer the P200 material, the more it modifies the asphalt binder and stiffens the HMA mix. The model also indicates that the higher the MBV (another indication of the presence of very fine P200), the higher is G*/sinδ. Methylene blue was the primary independent variable affecting the G*/sinδ of the F/A binder system.

m value (slope of the frequency versus G plot) at high temperature.* High m values indicate increasing rate of rut development in HMA mixes. The two-variable model (R^2=0.95, P=0.01) gives D60 as the primary independent variable and methylene blue as the secondary independent variable affecting m (see Table 72 and Figure 26). This is similar to G*/sinδ @ 0.1 Hz.

It is recommended to use D60 and methylene blue as the P200 tests which are related to HMA performance in terms of permanent deformation.

Fatigue Cracking

*G*sinδ @ 1.0 Hz at intermediate temperature.* High values of G*sinδ indicate high mixture stiffness at intermedi-

TABLE 72 Regression equations between P200 aggregate tests and HMA validation tests

0.8 F/A Gradation						
Performance Parameter	Step	Dependent	Independent	Equation	R^2	P
Permanent Deformation	1	G*/sinδ @0.1hz	German Filler	G*/sinδ=6845.29+59.737(German Filler)	0.43	0.16
Permanent Deformation	2	G*/sinδ @0.1hz	Rigden Voids (Penn State)	G*/sinδ=-8062.28+217.88(Rigden Voids, Penn State)+144.60(German Filer)	0.52	0.32
Permanent Deformation	1	m	German Filler	m=0.457-0.000699(German Filler)	0.42	0.17
Permanent Deformation	2	m	D10	m=0.450+0.0076(D10)-0.000876(German Filler)	0.65	0.21
Fatigue	1	G*sinδ @1.0hz	Rigden Voids (British Standard)	G*sinδ=120620.9-760.41(Rigden Voids ,British Standard)	0.31	0.25
Fatigue	2	G*sinδ @1.0hz	Methylene Blue	G*sinδ=123483.4–906.9(Rigden Voids ,British Standard)+457.7(Methylene Blue)	0.71	0.15
1.5 F/A Gradation						
Performance Parameter	Step	Dependent	Independent	Equation	R^2	P
Permanent Deformation	1	G*/sinδ @0.1hz	D60	G*/sinδ=28961.36-387.56(D60)	0.85	0.09
Permanent Deformation	2	G*/sinδ @0.1hz	Methylene Blue	G*/sinδ=25596.17-338.18(D60)+321.94(Methylene Blue)	0.94	0.015
Permanent Deformation	1	m	D60	m=0.256+0.003(D60)	0.90	0.004
Permanent Deformation	2	m	Methylene Blue	m=0.275+0.0027(D60)-0.0019(Methylene Blue)	0.95	0.01
Fatigue	1	G*sinδ @1.0hz	Methylene Blue	G*sinδ=89153.7+981.26(Methylene Blue)	0.60	0.07
Fatigue	2	G*sinδ @1.0hz	Rigden Voids (British Standard)	G*sinδ=124586.23-965.21(Rigden Voids,British Standard)+1064.7(Methylene Blue)	0.75	0.12

ate temperature and, therefore, increased fatigue cracking. The two-variable model for G*sinδ @ 1.0 Hz indicates methylene blue as the primary independent variable and Rigden voids (British standard) as the secondary variable. This is similar to the rutting models in that higher values of methylene blue indicate stiffer HMA mixes. However, the model has an R^2 value of 0.75 and a level of significance of 0.12 (greater than the desired 0.05). Therefore, it appears that the effect of P200 material at a 1.5 F/A ratio is statistically not significant and, therefore, no P200 test is recommended for fatigue cracking. Methylene blue was also the primary independent variable affecting the G*sinδ (fatigue factor) of the F/A binder system. German filler was the secondary independent variable in the F/A binder system models. German filler is related to Rigden voids (British standard) with a correlation coefficient (R) of −0.75 and a P value of 0.09.

Stripping

Two mix validation tests—AASHTO T 283 (Modified Lottman) and Hamburg wheel tracking device—were used to determine HMA performance in terms of resistance to stripping or moisture susceptibility, which may, in turn, cause permanent deformation.

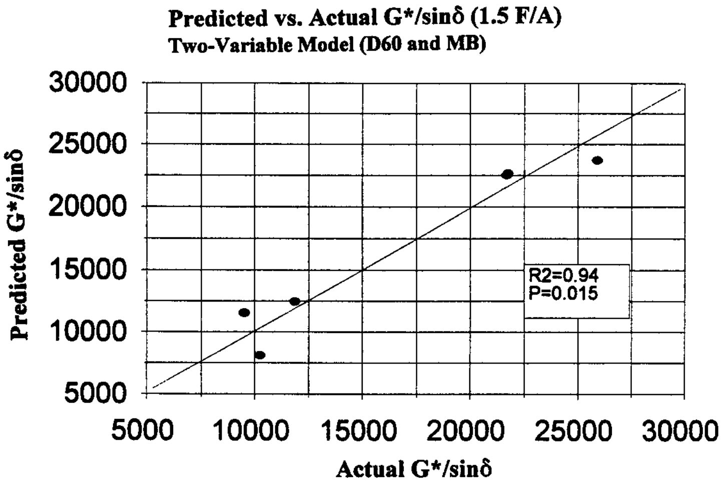


Figure 25. Predicted vs. actual G/sinδ (1.5 F/A).*

AASHTO T 283. Higher TSR obtained by this test indicates increased resistance to stripping. The two-variable model for TSR at an F/A ratio of 1.5 (R^2=0.82, P=0.08) consists of D10 (P200 size at 10 percent passing) as the primary independent variable (R^2=0.68, P=0.04) and specific surface area (SA) of P200 as the secondary independent variable (Table 73). TSR increases as the P200 becomes finer at 10 percent passing (D10 decreases). It appears that very fine size P200 at 10 percent passing level is stiffening the F/A binder and thus providing increased resistance to stripping. The literature review has indicated that high viscosity asphalt binders offer greater resistance to stripping than low-viscosity asphalt binders, all other things being equal.

The two-variable model (see Table 73) for TSR obtained at a 0.8 F/A ratio is much better than that obtained at a 1.5 F/A ratio. It has an R^2 value of 0.98 (P=0.003) and it has D10 and methylene blue as the primary and secondary independent variables affecting HMA stripping. This indicates that the fineness (D10) of the material as well as the nature (methylene blue) of the P200 material affects HMA resistance to stripping (Figure 27). It appears that the two-variable model for TSR obtained at a 1.5 F/A ratio had a lower R^2 value and higher P value because large amounts of fines stiffened the asphalt binder too much and masked the effect of the nature of the fines.

On the basis of the TSR obtained by AASHTO T 283, D10 and methylene blue are the recommended P200 aggregate tests which are related to stripping of HMA mixes. As stated earlier, D10 indicates the fineness of the P200 material and methylene blue indicates both fineness and nature of the P200 material.

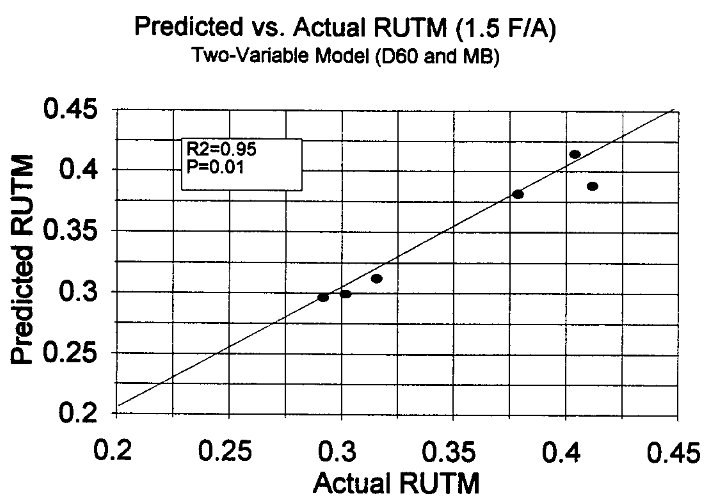


Figure 26. Predicted RUTM values vs. actual RUTM values using a two-variable (D60 and Methylene Blue) model.

TABLE 73 Regression equations between P200 aggregate tests and TSR

Performance Parameter	Step	Dependent	Independent	Equation	R^2	P
0.8 F/A Gradation						
Stripping	1	TSR	D10	TSR=71.21-4.844(D10)	0.94	0.002
Stripping	2	TSR	Methylene Blue	TSR=68.54-4.19(D10)+0.201(Methylene Blue)	0.98	0.003
1.5 F/A Gradation						
Performance Parameter	Step	Dependent	Independent	Equation	R^2	P
Stripping	1	TSR	D10	TSR=70.78-4.29(D10)	0.68	0.04
Stripping	2	TSR	Specific Surface Area	TSR=95.47-8.84(D10)-0.001(Specific Surface Area)	0.82	0.08

Hamburg Wheel Tracking. The inflection point obtained in this test represents the number of passes at which stripping starts to occur in the HMA mix. The larger the inflection point (number of passes), the higher is the mix's resistance to stripping. The two-variable model for inflection point at a 1.5 F/A ratio has a low R^2 (0.58) and high P value (0.28) and, therefore, is not considered statistically significant (Table 74). This model has methylene blue and specific surface area as the primary and secondary independent variables affecting stripping. The higher the methylene blue value, the lower the inflection point and hence resistance to stripping. This is in agreement with the observation made in the case of TSR obtained by AASHTO T 283.

The two-variable model for inflection point at a 0.8 F/A ratio is significantly better than that at a 1.5 F/A ratio (Table 74). This is in agreement with the trend seen in the case of TSR. The two-variable model (R^2=0.98, P=0.003) has D30 as the primary independent variable and Rigden voids (British Standard) as the secondary independent variable affecting resistance to stripping. D30 has a high correlation (R=0.99, P=0.0001) with D10 which was selected as the primary independent variable in the case of TSR. It is not understood why the Rigden voids (British Standard) was selected as the secondary independent variable by the statistical analysis. Normally, higher Rigden voids cause stiffer F/A systems and, therefore, should result in increased resistance to stripping (or higher values of inflection point). However, the model shows an opposite effect, because the slope of regression is negative.

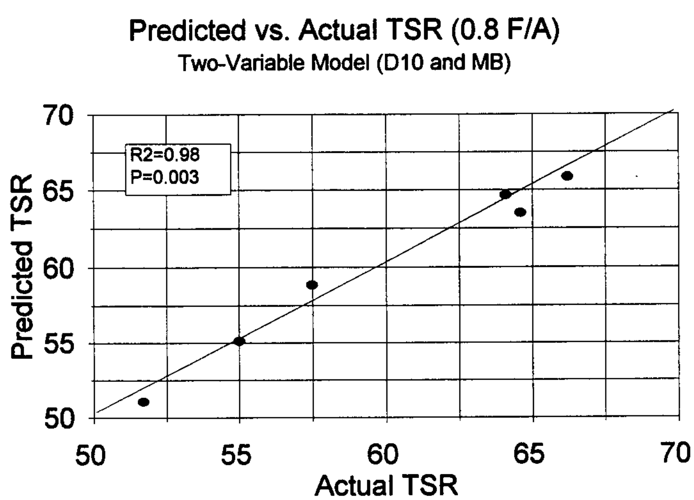


Figure 27. Predicted TSR values vs. actual TSR values for the two-variable (D10 and Methylene Blue) model.

TABLE 74 Regression equations between P200 aggregates tests and inflection point

0.8 F/A Gradation						
Performance Parameter	Step	Dependent	Independent	Equation	R^2	P
Rutting	1	Inflection Point	D30	Inflection Point = 5496.7+478.4(D30)	0.59	0.076
Rutting	2	Inflection Point	Rigden Voids British Standard	Inflection Point = 35183.4+458.4(D30)-792.2 (Rigden Voids British Standard)	0.98	0.003
1.5 F/A Gradation						
Performance Parameter	Step	Dependent	Independent	Equation	R^2	P
Rutting	1	Inflection Point	Methylene Blue	Inflection Point = 9784.9-268.6 (Methylene Blue)	0.31	0.25
Rutting	2	Inflection Point	Surface Area	Inflection Point = 5792.6-359.9 (Methylene Blue) +0.39 (Surface Area)	0.58	0.28

AASHTO T 283 and the Hamburg wheel tracking device are significantly different stripping tests. AASHTO T 283 involves curing of the mix in a 60°C oven for 16 hr followed by vacuum saturation of HMA, then immersion in a hot (60°C) water bath for 24 hr, transfer to 25°C water bath for 2 hr, and then testing for tensile strength. Hamburg wheel tracking does not involve any vacuum saturation. It does involve immersion of the HMA in a 50°C water bath. However, while immersed, the sample is subjected to the loaded wheel tracking device and rut depth measurements are taken. A repeated, dynamic mechanical load is applied to the HMA in Hamburg wheel tracking device whereas no such loading is applied in AASHTO T 283.

The conditioning and testing of HMA is significantly different in these two tests. Therefore, it is not surprising that different independent variables were selected in these two mix validation tests.

It is recommended that D10 and methylene blue be used for HMA performance in terms of stripping, taking into consideration both AASHTO T 283 and Hamburg wheel tracking tests.

D10 is the primary independent variable in case of TSR. It has high correlation with D30 (R=0.992, P=0.0001), which was selected as the primary independent variable in the Hamburg wheel tracking test.

Methylene blue is the secondary independent variable in the case of TSR. This test indicates the nature and fineness of the P200 material. This test was also selected in the case of "plastic fines in the fine aggregate" element of this project.

Conclusion

Table 75 shows the tests that are related to HMA performance based on the preceding statistical analysis and, therefore, recommended for evaluating aggregates for HMA in pavements.

PROPOSED SET OF AGGREGATE TESTS RELATED TO HMA PERFORMANCE

Table 76 summarizes the findings in terms of HMA performance parameters, related aggregate properties, and related aggregate tests. The information in Table 76 can also be presented as a list of nine aggregate tests which are related to various HMA performance parameters as shown in Table 77.

TABLE 75 Summary of recommended P200 tests

Performance Parameter	Recommended P200 Tests
Permanent Deformation	(1) D60 and Methylene Blue[1] (2) D10 and Methylene Blue[2]
Fatigue Cracking	none

TABLE 76 HMA performance parameters, related aggregate properties, and related aggregate tests

HMA Performance Parameter	Related Aggregate Properties	Related Aggregate Tests	Remarks
Permanent Deformation	Gradation and Size	Sieve Analysis (AASHTO T27)	Although gradation and size was not included in the research plan, they do affect permanent deformation.
	Coarse Aggregate Particle Shape and Surface Texture	(1) Uncompacted Void Content	This aggregate test is similar to AASHTO TP 33 which is for the fine aggregate. A proportionately larger equipment is used.
		(2) Flat or Elongated Particles in Coarse Aggregate (ASTM D 4791)	Flat or elongated particles (2:1 ratio) recommended although the ASTM D 4791 also measures flat and elongated particles.
	Fine Aggregate Particle Shape and Surface Texture	Uncompacted Void Content (AASHTO T304)	
	Properties of P200 Material[1]	(1) Particle Size Analysis	The particle sizes D60 and D10 corresponding to 60 and 10 percent passing, respectively, need to be measured. D60 contributes to permanent deformation directly whereas D10 causes permanent deformation through stripping.
		(2) Methylene Blue Test	Methylene Blue contributes to permanent deformation directly and indirectly (through stripping).
	Plastic Fines in the Fine Aggregate[2]	Methylene Blue Test	
Raveling, Popouts or Potholing[3]	Toughness/Abrasion Resistance and Durability/Soundness	(1) Micro-Deval Test	
		(2) Soundness of Aggregates by Use of Sodium and Magnesium Sulfate (AASHTO T104)	Although AASHTO T104 uses both Sodium and Magnesium Sulfate, only Magnesium Sulfate is recommended.
Fatigue Cracking	Gradation and Size	Size Analysis (AASHTO T27)	Although gradation and size was not included in the research plan, they do affect fatigue cracking.
	Coarse Aggregate Particle Shape and Surface Texture	(1) Uncompacted Void Content	This aggregate test is similar to AASHTO TP 33 which is for the fine aggregate. A proportionately larger equipment is used.
		(2) Flat or elongated Particles in Coarse Aggregates (ASTM D4791)	Flat or elongated particles (2:1 ratio) recommended although the ASTM D4791 also measures flat and elongated particles.

[1] P200 materials may cause permanent deformation of HMA by extending asphalt binder and/or inducing stripping in the HMA.
[2] Plastic fines in the fine aggregate may cause permanent deformation of HMA by inducing stripping in the HMA.
[3] These are HMA pavement distresses which can result from the use of soft or unsound aggregates.

TABLE 77 Set of aggregate tests related to HMA performance in pavements

Aggregate Tests	Related HMA Performance Parameters
1. Sieve Analysis for Determining Aggregate Gradation and Size (AASHTO T27)	Permanent deformation and fatigue cracking
2. Uncompacted Void content of Coarse Aggregate	Permanent deformation and fatigue cracking
3. Flat or Elongated Particles (2:1 ratio) in Coarse Aggregate	Permanent deformation and fatigue cracking
4. Uncompacted Void Content of Fine Aggregate (AASHTO TP33)	Permanent deformation
5. Methylene Blue Test of Fine Aggregate	Permanent deformation[1]
6. Particle Size Analysis of P200 Materials for Determining D60 and D10 Sizes	Permanent deformation[1]
7. Methylene Blue Test of P200 Material	Permanent deformation[1]
8. Micro-Deval Test	Raveling, popouts or potholing
9. Magnesium Sulfate Soundness Test	Raveling, popouts or potholing

[1]Permanent deformation may occur due to stripping.

CHAPTER 5

INTERPRETATION, APPRAISAL, AND APPLICATION

The primary objective of this project was to identify and recommend aggregate tests which are related to performance of HMA in pavements and should be used for evaluating aggregates for HMA. The interpretation, appraisal, and application of the nine aggregate test methods which have been recommended follow.

GRADATION AND SIZE

Gradation and size are related to permanent deformation and fatigue cracking of HMA on the basis of the review of literature. However, the effect of gradation and size on these HMA performance parameters has not been adequately quantified. A standard sieve analysis test (AASHTO T 27) is available and is widely used for determining the gradation and size of aggregates.

UNCOMPACTED VOID CONTENT OF COARSE AGGREGATE

UV content of coarse aggregate is related to permanent deformation and fatigue cracking of HMA. The higher the UV content, the greater the resistance of HMA to permanent deformation and fatigue cracking. This test is similar to the UV content of fine aggregate (AASHTO T 304), but is conducted with proportionally larger equipment. The proposed test method in AASHTO format is given in Appendix D. This test should replace the test for measuring the percentage of fractured particles in coarse aggregate (ASTM D 5821). Highway agencies can specify a minimum percentage of UV rather than the minimum percentage of one-face or two-face fractured particles. Further research is needed to validate this test method in the field and establish minimum desirable values of UV (see Chapter 6). If the HMA is subjected to Superpave mix analysis and is found to meet the permanent deformation and fatigue cracking criteria, then minimum UV values need not be specified.

FLAT OR ELONGATED PARTICLES (2:1 RATIO) IN COARSE AGGREGATE

The percentage of flat *or* elongated particles (2:1 ratio) is related to permanent deformation and fatigue cracking of HMA. Low percentages are desirable for improved performance. The test is conducted in accordance with ASTM D 4791 and should be adopted by AASHTO. The percentage of flat *or* elongated particles should be used in lieu of the percentage of flat *and* elongated particles (on the basis of the ratio of maximum to minimum dimensions of the particle) which is most commonly used in the United States. About 80 percent of the states specify a ratio of 5:1 for flat *and* elongated particle (*5*). Some states specify 4:1 and 3:1 ratios for flat *and* elongated particles. Two of the three regression models for rutting in this study selected the percentage of flat *or* elongated particles (2:1 ratio) as an important secondary variable, UV being the primary independent variable. It is quite likely that the 2:1 ratio was selected rather than 3:1 or 5:1 ratios because this ratio is closer to a cubical shape and yields higher numerical values for the model. Obviously, the maximum allowable percentage of flat *or* elongated particles (2:1 ratio) will be significantly higher than those of flat *or* elongated particles (3:1 and 5:1 ratios). The maximum permissible values need to be established through laboratory and field experiments. None of the rutting or fatigue models selected the percentage of flat *and* elongated particles at any ratio; this is logical because the test for flat *and* elongated particles measures neither flat particles nor elongated particles, it simply measures the ratio between the length and thickness of the particles. Therefore, the highway agencies must switch from flat *and* elongated particles to flat *or* elongated particles.

UNCOMPACTED VOID CONTENT OF FINE AGGREGATE

The UV content of fine aggregate is related to permanent deformation of HMA. The higher the UV content, the greater the resistance of HMA to permanent deformation. This test has been recommended in Superpave and has been adopted by AASHTO (T 304). The minimum permissible UV of the fine aggregate should replace the current specifications of specifying maximum permissible amounts of natural sand in HMA mixes. The minimum permissible UV values need to be established through field experiments. Minimum UV values need not be specified if the HMA mix meets the design criteria for permanent deformation when subjected to Superpave mix analysis.

METHYLENE BLUE TEST OF FINE AGGREGATE

The methylene blue test of fine aggregate is related to stripping of HMA mixtures, which in turn leads to permanent deformation of HMA pavements. The proposed test method in AASHTO format is given in Appendix D. This test should replace the sand equivalent test and plasticity index test, which are currently used in the United States for minimizing the stripping problem in HMA (*5*). The methylene blue test evaluates the amount and nature of deleterious fines in the fine aggregate, which may cause stripping. The higher the methylene blue index (MBI), the more susceptible the HMA mix is to stripping. The maximum permissible MBI values need to be established through laboratory and field experiments. However, the fine aggregate should not be rejected on account of its high MBI value because antistripping agents, both liquid as well as hydrated lime, are available to mitigate the stripping problem. If the finished HMA mix meets the test criteria of stripping tests (e.g., AASHTO T 283 and the Hamburg wheel tracking tests), it should be accepted. The methylene blue test in this study was conducted on the P200 material obtained from the fine aggregate by dry seiving. The adherent fines also need to be tested. The methylene blue test has also been attempted in Europe on the fine aggregate retained on a 75 μm sieve. Further laboratory research needs to be done to develop a procedure for testing the entire fine aggregate, if possible.

PARTICLE SIZE ANALYSIS OF P200 MATERIAL

An automated device for measuring the gradation of the P200 material was used in this study. D60 and D10, which are particle sizes in microns corresponding to 60 and 10 percent passing, respectively, are related to performance of HMA. D60 and D10 are obtained from the gradation plot (particle size in microns versus percent passing). D60 is related to permanent deformation of HMA caused directly by traffic load applications—the lower the D60 value, the greater the HMA resistance to permanent deformation. D10 is related to stripping of HMA which, in turn, may cause permanent deformation of HMA—the lower the D10 value, the higher the HMA resistance to stripping. Although the gradation of the P200 material can be obtained by AASHTO T 88 with a hydrometer, it is recommended to use an automated device similar to the one used in this study. A test procedure in AASHTO format is included in Appendix D. Appropriate values for specification need to be established for D60 and D10 based on laboratory and field experiments (see Chapter 6).

METHYLENE BLUE TEST OF P200 MATERIAL

The methylene blue test conducted on the P200 material (such as baghouse fines and fillers) is related to stripping of HMA, which, in turn, may cause permanent deformation of HMA pavement. The MBI value indicates the nature and amount of detrimental clay type material present in the P200 material—the higher the MBI value, the lower the HMA resistance to stripping. The methylene blue test in AASHTO format is included in Appendix D. Maximum permissible values of the MBI need to be established through laboratory and field experiments. However, maximum MBI values need not be specified if the finished HMA mix is tested for moisture-induced damage (such as AASHTO T 283) and meets the specified criteria.

MICRO-DEVAL AND MAGNESIUM SULFATE SOUNDNESS TESTS

Currently, 94 percent of the states surveyed use the Los Angeles abrasion test (AASHTO T 96) for evaluating toughness and abrasion resistance of coarse aggregate (*5*). Fifty-four percent of the states use sodium sulfate soundness, and 19 percent of the states use magnesium sulfate soundness for evaluating the durability and soundness of the coarse aggregate (*5*). Micro-Deval tests and magnesium sulfate soundness tests are related to the performance of HMA in terms of raveling, popouts, or potholing. These two tests were able to separate good and fair aggregates from poor aggregates in this study. Therefore, these two tests have been recommended in lieu of the Los Angeles abrasion test, sodium sulfate soundness test, and other soundness tests (e.g., freeze-thaw loss and durability index). Appendix D contains the Micro-Deval test in AASHTO format. The magnesium sulfate soundness should be conducted in accordance with AASHTO T 104. The magnesium sulfate soundness test has a lower testing variability compared with the sodium sulfate soundness tests. Field experimental test sections are needed to establish maximum allowable loss values for Micro-Deval and magnesium sulfate soundness tests for different climatic zones in the United States (see Chapter 6).

CHAPTER 6

CONCLUSIONS AND SUGGESTED RESEARCH

CONCLUSIONS

The following conclusions can be drawn from this study.

1. The following performance parameters of HMA used in pavement construction are affected by the properties of aggregates:

    - Permanent deformation (resulting directly from traffic loading as well as indirectly after HMA has stripped);
    - Raveling, popouts, or potholing;
    - Fatigue cracking; and
    - Frictional resistance.

The preceding performance parameters, except frictional resistance, were studied in this research project.

2. The following aggregate properties influence the performance parameters identified in item 1 above:

    - Gradation and size;
    - Particle shape, angularity, and surface texture;
    - Porosity or absorption;
    - Cleanliness and deleterious materials;
    - Toughness and abrasion resistance;
    - Durability and soundness;
    - Expansive characteristics;
    - Polish and frictional characteristics;
    - Mineralogy and petrography; and
    - Chemical properties.

The following aggregate properties were studied in this project: particle shape, angularity, and surface texture; plastic fines in fine aggregate; toughness/abrasion resistance; durability/soundness; and characteristics of P200 material.

3. The following nine aggregate tests are related to performance of HMA in pavements in terms of permanent deformation; fatigue cracking; and raveling, popout, and potholing as indicated in parentheses (frictional resistance was not considered). These aggregate tests are recommended for evaluating aggregates for HMA in pavements.

    - Sieve Analysis of Aggregates for Determining Gradation and Size (permanent deformation and fatigue cracking);
    - Uncompacted Void Content of Coarse Aggregate (permanent deformation and fatigue cracking);
    - Flat or Elongated Particles (2:1 ratio) in Coarse Aggregate (permanent deformation and fatigue cracking);
    - Uncompacted Void Content of Fine Aggregate (permanent deformation);
    - Methylene Blue test of Fine Aggregate (permanent deformation resulting from stripping);
    - Particle Size Analysis of P200 Material for Determining D60 and D10 sizes (permanent deformation resulting from traffic loads as well as stripping);
    - Methylene Blue test of P200 Material (permanent deformation resulting from stripping);
    - Micro-Deval test (raveling, popouts, or potholing); and
    - Magnesium Sulfate Soundness test (raveling, popouts, or potholing).

4. The aggregate tests given in item 3, which have been recommended primarily on the basis of laboratory experiments, need to be validated in the field. This can be accomplished by monitoring the performance of various HMA text sections incorporating aggregates with a wide range of properties.

SUGGESTED RESEARCH AND FIELD VALIDATION PLAN

The effect of gradation and size of the aggregate on permanent deformation or fatigue cracking of HMA pavements was not researched in this project; however, this needs to be done to recommend optimum gradations for dense- and open-graded mixes.

Free mica in the fine aggregate has been reported to be detrimental to the performance of HMA mixes. Research is needed to develop a suitable test method for determining free mica content and evaluating its effect on HMA performance.

The following field validation plan is suggested to validate the recommended performance-related aggregate tests.

Uncompacted Void Content of Coarse Aggregate and Flat or Elongated Particles (2:1 ratio) in Coarse Aggregate

Use a test track or accelerated loading facility (ALF) to evaluate permanent deformation and fatigue cracking of at least eight HMA test sections. Use a common fine aggregate (preferably a natural sand) in all test sections to accentuate the effect of eight different coarse aggregates. Select a common gradation for all HMA mixes, preferably below the maximum density line, to maximize the effect of the coarse aggregate. Select coarse aggregates of different particle shape, angularity, and surface texture (ranging from round smooth gravel to angular crushed stone). If an ALF is used, select a state where a wide variety of aggregates are available for constructing test sections. Measure the UV and the percentage of flat or elongated particles (2:1 ratio) in all coarse aggregates and evaluate their relationship to permanent deformation and fatigue cracking of the test sections.

Uncompacted Void Content of Fine Aggregate

A field validation plan similar to the preceding plan for coarse aggregates is recommended. Construct at least eight different sections using different fine aggregates ranging in particle shape, angularity, and surface texture (ranging from round natural sand to angular crushed sand). Use a common coarse aggregate (preferably round gravel) and a mix gradation above the maximum density line to accentuate the effect of the different fine aggregates. Evaluate the permanent deformation and fatigue cracking of all test sections either on a test track or by an ALF. Determine the relationship between UV and permanent deformation and fatigue cracking of HMA pavements.

Methylene Blue Test of Fine Aggregate

Construct at least eight test sections on a test track or to be tested by an ALF. Use a common coarse aggregate (preferably a limestone with no history of stripping) and eight different fine aggregates of different mineralogical compositions and a wide range of MBI values. The HMA gradation and relative proportions of the fine aggregate and coarse aggregate should be held reasonably consistent. To accelerate the stripping phenomenon it is necessary to provide an inadequate subsurface drainage system under all eight test sections. This can be accomplished by designing the pavement structure to create the so-called "bath tub" effect which will keep the HMA test sections reasonably saturated with water during load applications. Obtain cores from the test pavements periodically and measure the retained modulus and tensile strength and examine the extent of stripping. Evaluate the relationship between MB values and the loss of HMA strength because of stripping and the extent of stripping.

Particle Size Analysis and Methylene Blue of P200 Material

The field validation plan to validate the effect of D60, D10, and MBI on permanent deformation is likely to be cumbersome and costly. Ideally, it would require the use of a common coarse aggregate and a common fine aggregate and different types of P200 materials (baghouse fines and mineral fillers). The P200 materials should have a wide range of D60, D10, and MBI values. These different P200 materials have to be incorporated in washed coarse and fine aggregates. Test sections should be subjected to traffic loads either on a test track or by an ALF. Evaluate the effect of D60, D10, and MBI values on permanent deformation of HMA test sections.

Micro-Deval Test and Magnesium Sulfate Soundness Test

Select one state in each of the four climatic zones in the United States (wet and freeze, wet and no freeze, dry and freeze, and dry and no freeze). The selected states should have a wide variety of aggregates of different mineralogical compositions and field performance history ranging from good to poor. Both Micro-Deval and magnesium sulfate soundness tests should be conducted on a large number of available aggregates to select aggregates which give a wide range of test values. Construct HMA test sections in each of the four selected states using these different aggregates. At the time of construction, determine the gradation of the aggregate at the following stages: (a) aggregate in stockpile in the HMA plant, (b) aggregate in cold feed (belt sample), (c) aggregate extracted from loose HMA mix just after mixing, and (d) aggregate extracted from HMA mat after compaction. The preceding test data should be used to quantify degradation of aggregate resulting from handling, mixing, and compaction. Results from the Micro-Deval test should be correlated with the aggregate degradation occurring during HMA construction.

The surface of the HMA test sections constructed with different aggregates should be visually evaluated annually (for at least 5 years) for raveling, popouts, or potholing. A team of material engineers should evaluate the surface for these distresses resulting from the action of traffic and environment. The subjective ratings obtained by the team should be quantified and averaged. Both Micro-Deval and magnesium sulfate soundness test results should be correlated with the average surface evaluation ratings.

The preceding field validation plans will not only help validate the recommended aggregate tests and will be helpful in establishing minimum and/or maximum test values for specification purposes.

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APPENDIXES A, B, C AND E

UNPUBLISHED MATERIAL

Appendixes A, B, C, and E contained in the research agency's final report are not published herein. For a limited time, copies of the report, "Aggregate Tests Related to Asphalt Concrete Performance in Pavements," containing these appendixes will be available on a loan basis or for purchase ($22.00) on request to NCHRP, Transportation Research Board, Box 289, Washington, D.C., 20055. The available appendixes are as follows:

Appendix A: Review of Literature on Aggregate Properties Related to HMA Performance Parameters

Appendix B: Petrographic Reports for 16 Aggregates Used in Toughness/Abrasion Resistance and Durability/Soundness Study

Appendix C: Performance Questionnaire for Aggregates with Fair and Poor Performance Relative to Toughness and Durability

Appendix E: *Transportation Research Circular 479* "Aggregate Tests for Hot-Mix Asphalt: State of the Practice"

APPENDIX D

RECOMMENDED NEW AGGREGATE TESTS

These proposed testing methods are the recommendation of NCHRP Project 4-19 staff at Auburn University. These methods have not been approved by NCHRP or any AASHTO committee or formally accepted for the AASHTO specifications.

PROPOSED STANDARD METHOD OF TEST FOR UNCOMPACTED VOID CONTENT OF COARSE AGGREGATE (AS INFLUENCED BY PARTICLE SHAPE, SURFACE TEXTURE, AND GRADING)

1. Scope

1.1 This method describes the determination of the loose uncompacted void content of a sample of coarse aggregate. When measured on any aggregate of a known grading, void content provides an indication of the aggregates' angularity, sphericity, and surface texture compared with other coarse aggregates tested in the same grading.

1.2 Three procedures are included for the measurement of void content. Two use graded coarse aggregate (standard grading or as-received grading), and the other uses several individual size fractions for void content determinations.

1.2.1 Standard Graded Sample (Method A)—This method uses a standard coarse aggregate grading that is obtained by combining individual sieve fractions from the maximum density curve drawn from the maximum coarse aggregate size. See the section on Preparation of Test Samples for the grading.

1.2.2 Individual Size Fractions (Method B)—This method uses each of three coarse aggregate size fractions: (a) 19 mm (¾″) to 12.5 mm (½″); (b) 12.5 mm (½″) to 9.5 mm (⅜″); and (c) 9.5 mm (⅜″) to 4.75 mm (No. 4). For this method, each size is tested separately.

1.2.3 As-Received Grading (Method C)—This method uses that portion of the coarse aggregate retained on the 4.75 mm (No. 4) sieve.

1.3 The values stated in SI units shall be regarded as the standard.

1.4 This standard does not purport to address all of the safety problems, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulation limitations prior to its use.

2. Referenced Documents

2.1 AASHTO Standards

T19 Test Method for Unit Weight and Voids in Aggregate

T11 Test Method for Materials Finer than 75-µm (No. 200) Sieve in Mineral Aggregates by Washing

T85 Test Method for Specific Gravity and Absorption of Coarse Aggregate

T27 Test Method for Sieve Analysis of Fine and Coarse Aggregates

T248 Practice for Reducing Field Samples of Aggregate to Testing Size

T2 Practice for Sampling Aggregates

3. Summary of Test Method

3.1 A calibrated cylindrical measure is filled with coarse aggregate of prescribed grading by allowing the sample to flow through a funnel from a fixed height into the measure. The coarse aggregate is struck off, and its mass is determined by weighing. Uncompacted void content is calculated as the difference between the volume of the cylindrical measure and the absolute volume of the coarse aggregate collected in the measure. Uncompacted void content is calculated using the bulk dry specific gravity of the coarse aggregate. Two runs are made on each sample and the results are averaged.

3.1.1 For a graded sample (Test Method A or C), the percent void content is determined directly, and the average value from two runs is reported.

3.1.2 For the individual size fractions (Test Method B), the mean percent void content is calculated using the results from tests of each of the three individual size fractions.

4. Significance and Use

4.1 Methods A and B provide percent void content determined under standardized conditions which depends on the particle shape and texture of a coarse aggregate. An increase in void content by these procedures indicates greater angularity, less sphericity, or rougher surface texture, or some combination of the three factors. A decrease in void content results is associated with more rounded, spherical, smooth-surfaced coarse aggregate or a combination of these factors.

4.2 Method C measures the uncompacted void content of the plus 4.75 mm (No. 4) portion of the as-received material. This void content depends on grading as well as particle shape and texture.

4.3 The void content determined on the standard graded sample (Method A) is not directly comparable with the average void content of the three individual size fractions from the same sample tested separately (Method B). A sample consisting of single-sized particles will have a higher void content than a graded sample. Therefore, use either one method or the other as a comparative measure of shape and texture, and identify which method has been used to obtain the reported data. Method C does not provide an indication of shape and texture directly if the grading from sample to sample changes.

4.3.1 The bulk dry specific gravity of the coarse aggregate is used in calculating the void content. The effectiveness of these methods of determining void content and its relationship to particle shape and texture depends on the bulk specific gravity of various size fractions being equal, or nearly so. The void content is actually a function of the volume of each size fraction. If the type of rock or minerals, or its porosity, in any of the size fractions varies markedly, it may be necessary to determine the specific gravity of the size fractions used in the test.

5. Apparatus

5.1 Cylindrical Measure—A cylindrical metal measure shall be watertight with the top and bottom true and even, preferably machined to accurate dimensions on the inside, and sufficiently rigid to retain its form under rough usage. The top rim shall be smooth and plane within 0.25 mm and shall be parallel to the bottom within 0.5 deg. The inside diameter shall be approximately 155 mm and the inside height shall be approximately 160 mm. See Figure D-1.

5.2 Funnel—The lateral surface of the right frustum of a cone sloped 60 ± 4 deg from the horizontal with an opening of 105 ± 2mm diameter. The funnel section shall be a piece of metal, smooth on the inside. It shall have a value of at least two times the volume of the cylindrical measure or shall be provided with a supplemental metal container to provide the required volume. See Figure D-1.

5.3 Funnel stand—A support capable of holding the funnel firmly in position with the axis of the funnel colinear (within a 4-deg angle and a displacement of 2 mm) with the axis of the cylindrical measure. The funnel opening shall be 115 ± 2 mm above the top of the cylinder.

5.4 Glass Plate—A square glass plate approximately 170 mm × 170 mm with a minimum thickness of 4 mm used to calibrate the cylindrical measure.

5.5 Pan—A metal or glass pan of sufficient size to contain the funnel stand and to prevent loss of material. The purpose of the pan is to catch and retain aggregate particles that overflow the measure during filling and strike off.

5.6 Flat metal rod—A flat metal rod of 300 ± 5 mm in length, 40 ± 2 mm in width, and 3-mm thickness is used to strike off the top of the container.

5.7 Scale or balance accurate and readable to ± 0.1g within the range of use, capable of weighing the cylindrical measure and its contents.

6. Sampling

6.1 The samples used for this test shall be obtained using Practice T2 and Practice T248 or from sieve analysis samples used for Test Method T27. For methods A and B, the sample is washed over a 75 μm (No. 200) sieve in accordance with Test Method T11 and then dried and sieved into separate size fractions according to Test Method T27 procedures. Maintain the necessary size fractions obtained from one (or more) sieve analysis in a dry condition in separate containers for each size. For method C, dry and split the as-received sample in accordance with the drying procedure in Test Method T27.

7. Calibration of Cylindrical Measure

7.1 Apply a light coat of grease to the top edge of the dry, empty cylindrical measure. Weigh the measure, grease, and glass plate. Fill the measure with freshly boiled, deionized water at a temperature of 18 to 24°C. Record the temperature of the water. Place the glass plate on the measure, being sure that no air bubbles remain. Dry the outer surfaces of the measure and determine the combined mass of measure, glass plate, grease, and water by weighing. Following the final

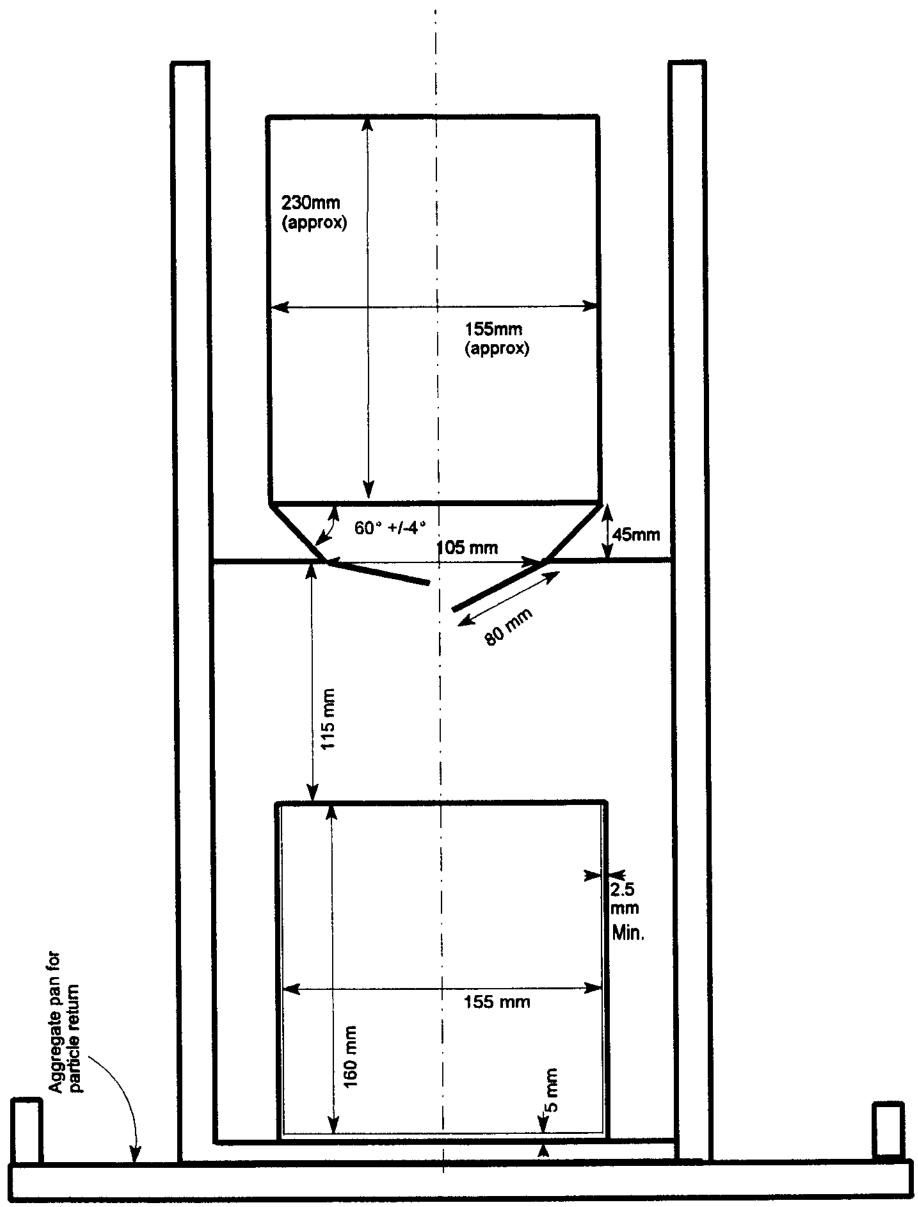

Figure D-1. Test apparatus.

weighing, remove the grease, and determine the mass of the clean, dry, empty measure for subsequent tests.

7.2 Calculate the volume of measure as follows:

where:

$V = 1000(M/D)$

V = Volume of cylinder, mL
M = net mass of water, g
D = density of water (see table in T19 for density at the temperature used), kg/m^3

Determine the volume to the nearest 0.1 mL.

8. Preparation of Test Samples

8.1 Method A—Standard Graded Sample—weigh out and combine the following quantities of coarse aggregate (based on the maximum size of the aggregate) which has been dried and sieved in accordance with Test Method T27. (See Table D-1). The total sample weight should be 5000 g ± 10g.

8.2 Method B—Individual Size Fractions—Prepare a separate 5000 g sample of coarse aggregate, dried and sieved in

TABLE D-1 Standard Gradations for Method A

Maximum Size of Aggregate	Sieve Size (mm)	Mass, g
19mm	19mm (¾″) to 12.5mm (½″)	1740
	12.5mm (½″) to 9.5mm (⅜″)	1090
	9.5mm (⅜″) to 4.75mm (No.4)	2170
12.5mm	12.5mm (½″) to 9.5mm(⅜″)	1970
	9.5mm (⅜″) to 4.75mm (No. 4)	3030

accordance with Test Method T27, for each of the following size fractions: (See Table D-2).

8.3 Method C—As-Received Grading—Pass the sample (dried in accordance with Method T27) over a 4.75 mm (No. 4) sieve. Obtain a 5000 g ± 10g sample of the material.

The tolerance on each of the amounts in Table D-2 is 10 g. Do not mix these samples together. Each size is tested separately.

8.4 Specific Gravity of Coarse Aggregate—If the bulk dry specific gravity of coarse aggregate from the source is unknown, determine it on the plus 4.75 mm (No. 4) sample according to Test Method T85.

9. Procedure

9.1 Mix each test sample until it appears to be homogeneous. Position the cylindrical measure under the funnel section as shown in Figure 1. Pour the test sample into the funnel. Remove the rod from the bottom of the funnel to allow the aggregate to fall freely into the cylindrical measure.

9.2 After the funnel empties, strike off excess heaped aggregate from the cylindrical measure. Until this operation is complete, exercise care to avoid vibration or any disturbance that could cause compaction of the coarse aggregate in the cylindrical measure. Remove any aggregate that may have fallen on the outside of the container and determine the mass of the cylindrical measure and contents to the nearest 0.1g. Retain all aggregate particles for a second test run.

9.3 Recombine the sample from the retaining pan and cylindrical measure and repeat the procedure. The results of two runs are averaged (see Section 10).

TABLE D-2 Size Fractions for Method B

Sieve Size (mm)	Mass, g
19mm (¾″) to 12.5mm (½″)	5000
12.5mm (½″) to 9.5mm(⅜″)	5000
9.5mm (⅜″) to 4.75mm (No. 4)	5000

9.4 Record the mass of the empty measure. Also, for each run, record the mass of the measure and coarse aggregate.

10. Calculation

10.1 Calculate uncompacted voids for each determination as follows:

$$U = \frac{V - F/G}{V} * 100$$

V = volume of cylindrical measure, mL
F = net mass, g, of coarse aggregate in measure (Gross mass minus the mass of the empty measure)
G = bulk dry specific gravity of coarse aggregate
U = uncompacted voids, percent, in the material

10.2 For the Standard Graded Sample (Method A), calculate the average uncompacted voids for the two determinations and report the result as U_S.

10.3 For the Individual Size Fractions (Method B), calculate:

10.3.1 First, the average uncompacted voids for the determination made on each of the three size-fraction samples:

U_1 = Voids, 19 mm(¾″) to 12.5 mm(½″), percent

U_2 = Uncompacted Voids, 12.5 mm(½″) to 9.5 mm(⅜″), percent

U_3 = Uncompacted Voids, 9.5 mm(⅜″) to 4.75 mm(No. 4), percent

10.3.2 Second, the mean uncompacted voids (U_m), including the results for all three sizes:

$U_m = (U_1 + U_2 + U_3) / 3$

10.4 For the As-Received grading (Method C), calculate the average uncompacted voids for the two determinations and report the results as U_R.

11. Report

11.1 For the Standard Graded Sample (Method A):

11.1.1 The Uncompacted Voids (U_S) in percent to the nearest one-tenth of a percent.

11.1.2 The specific gravity value used in the calculation.

11.2 For the Individual Size Fractions (Method B), report the following percent voids to the nearest one tenth of a percent.

11.2.1 Uncompacted Voids for size fraction (***a***) 19 mm (³⁄₄″) to 12.5 mm (¹⁄₂″) (U_1), (***b***) 12.5 mm (¹⁄₂″) to 9.5 mm (³⁄₈″) (U_2), and (***c***) 9.5 mm (³⁄₈″) to 4.75 mm (No. 4) (U_3).

11.2.2 Mean Uncompacted Voids (U_m).

11.2.3 Specific gravity values used in the calculations, and whether the specific gravity values were determined on a graded sample or the individual size fractions used in the test.

11.3 For the As-Received Sample (Method C), report:

11.3.1 The uncompacted voids (U_R) in percent to the nearest ¹⁄₁₀th of a percent.

11.3.2 The specific gravity value used in the calculation.

12. Precision and Bias

12.1 Precision—No precision has been established.

12.2 Bias—Since there is no accepted reference material suitable for determining the bias for the procedures in these test methods, bias has not been determined.

13. Keywords—Angularity, Coarse Aggregate, Particle Shape, Surface Texture, Void Content

PROPOSED STANDARD METHOD OF TEST FOR METHYLENE BLUE VALUE OF CLAYS, MINERAL FILLERS, AND FINES

1. Scope

1.1 This standard provides procedures for determining the amount of potentially harmful fine material (including clay and organic material) present in an aggregate.

1.2 The values stated in SI units are to be regarded as the standard.

1.3 This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this procedure to establish appropriate safety and health practices and to determine the applicability of regulatory limitations prior to use.

2. Summary of Test

2.1 Methylene Blue solution is titrated into distilled water containing the sample material (passing 75 μm sieve) in increments. A small amount of water containing the sample material and titrated Methylene Blue is removed via a glass rod and dropped onto filter paper. When the sample aggregate can no longer absorb more Methylene Blue, a blue ring is formed on the filter paper.

3. Significance and Use

3.1 The Methylene Blue Value determined by this standard can be used to estimate the amount of harmful clays and organic matter present in an aggregate. A large value for Methylene Blue Value indicates a large amount of clay or organic material present in the sample.

4. Apparatus

4.1 Amber-colored burette of at least 50 ml capacity

4.2 Magnetic mixer with stir bar

4.3 Balance sensitive to 0.01 g

4.4 Glass rod of approximately 250 mm length and approximately 8 mm diameter

4.5 Timer or stop watch

4.6 Sieve (75 μm) and pan

4.7 Volumetric flask of 1000 ml capacity

4.8 Whatman No. 2 filter paper

4.9 3 glass beakers

4.10 Methylene Blue, reagent grade—dated and stored for no more than 4 months in a brown bottle wrapped with foil in a dark cabinet at lab temperature.

4.11 Distilled water at lab temperature.

5. Sampling

5.1 A representative sample of the fine aggregate to be tested is dried to constant weight and screened through the 75 μm (No. 200) sieve. The portion passing through the sieve (P200 material) is retained for testing; the rest is discarded.

6. Procedure

6.1 Weigh out 10.0 g (± 0.05g) of the P200 material that has been dried to constant weight and place in beaker.

6.2 Add 30 g of distilled water and stir with the mixer until the P200 material is uniformly dispersed.

6.3 One gram of Methylene Blue is dissolved in enough distilled water to produce 200 ml of solution, with each 1 ml of solution containing 5 mg of Methylene Blue.

6.4 With the slurry still mixing, fill the burette with the Methylene Blue solution, add 0.5 ml of the solution to the slurry, and stir for 1 minute.

6.5 Remove a drop of the slurry, using the glass stirring rod, and place on the filter paper.

6.6 Observe the appearance of the drop on the filter paper. The end point is indicated by the formation of a light blue halo around the drop. Continue adding the Methylene Blue solution to the slurry in 0.5 ml increments with 1 minute stirring after each addition, then testing, until the end point is reached.

6.7 After the end point is reached, continue stirring for 5 minutes and retest.

Note: With experience, the person performing the test can reach the end point more quickly by skipping early increments.

7. Calculation

7.1 $MBV = CV/W$

where

MBV = Methylene Blue Value in mg of solution per g of the P200 material.
C = mg of Methylene Blue/ml of solution
V = ml of Methylene Blue solution required for titration
W = grams of dry material

7.2 The calculations may be simplified by inserting the mg of Methylene Blue per ml of solution and the grams of dry material.

$$MBV = (5 \times V)/10$$

$$MBV = 0.5V$$

8. Precision and Bias

8.1 Precision—No precision has been established for this test.

8.2 Bias—No bias can be established because no reference material is available for this test.

PROPOSED STANDARD METHOD OF TEST FOR DETERMINATION OF PARTICLE SIZE DISTRIBUTION OF MINERAL FILLERS BY LASER DEVICE

1. Scope

1.1 This test method covers the determination of the particle size distribution of mineral filler using a laser particle size analyzer.

1.2 The values stated in SI units are to be regarded as the standard.

1.3 This method involves the use of a laser. The laser beam can cause eye damage if viewed either directly or indirectly from reflective surfaces (such as a mirror or shiny metal surface). Avoid direct exposure to the beam. The beam should never be viewed directly or with optical instruments. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this procedure to establish appropriate safety and health practices and to determine the applicability of regulatory limitations prior to use.

2. Referenced Documents

2.1 AASHTO Standards

M17	Mineral Filler for Bituminous Paving Mixtures
T2	Sampling Aggregates
T127	Sampling and Amount of Testing of Hydraulic Cement
T248	Reducing Field Samples of Aggregate to Testing Size

2.2 ASTM Standards

C50	Practice of Sampling, Inspection, Packing, and Marking of Lime and Limestone Products
C311	Test Methods for Sampling and Testing Fly Ash or Natural Pozzolans for Use as a Mineral Admixture in Portland Cement Concrete

C670 Preparing Precision and Bias Statements for Test Methods for Construction Materials

2.3 Particle Size Analysis equipment manufacturer's instruction manual

3. Summary of Method

3.1 Fluid suspensions of water, surfactant, and the mineral filler at the appropriate concentration are passed in front of a laser beam. The resulting light diffraction is detected and fitted to an optical model (Fraunhofer model has been found to be acceptable). The data output from the model is then used to determine the particle size distribution.

4. Significance and Use

4.1 This method is used to determine the gradation of mineral fillers proposed for use as aggregates or being used as aggregates. The results are used to determine compliance of the particle size distribution with applicable specifications requirements and to provide necessary data for control of the production of various aggregate products and mixtures containing aggregates.

5. Apparatus

5.1 Laser/Optical device—The test instrument shall utilize the Fraunhofer laser diffraction optical modeling and Mie scattering PIDS (Polarization Intensity Differential Scattering). The equipment shall be capable of measuring particles in the size range of 0.375 μm to 2000 μm. This must be accomplished in a single analysis scan. The equipment shall have a minimum of 92 size channels and a minimum of 126 detectors. The optical control shall be an in-line design with automatic alignment. Laser shall be solid state (750 nm).

5.2 50 ml glass beaker

5.3 Small spatula

5.4 Glass stirring rod

5.5 Surfactant (Aerosol OT solution has been found to be acceptable)

6. Sampling

6.1 Mineral Filler—Sample the mineral filler according to AASHTO T127, ASTM C 50 or ASTM C 311, whichever is most appropriate for the material being sampled.

6.2 The minimum size of field samples shall be 5.0 kg. Reduce the field sample to a minimum size of 100 g for testing.

7. Preparation of Test Specimens

7.1 Using a spatula, stir the sample thoroughly and put approximately 5 g of material into the small beaker.

7.2 Add approximately 20 ml of water and about 8 drops of the surfactant to the sample.

7.3 Stir the sample with the glass rod.

7.4 Rinse the rod of any material.

8. Procedure

8.1 Testing shall be conducted in accordance with the procedures outlined in the manufacturer's equipment manual.

9. Calculation

9.1 Calculate the Fineness Modulus as follows:

FM = (The sum of percentages of dust coarser than 75, 50, 30, 20, 10, 5, 3, and 1 microns)/100

10. Report

10.1 Report the following:

- Fineness Modulus
- D10 (Particle size, in mm, that has 10% passing)
- D30 (Particle size, in mm, that has 30% passing)
- D60 (Particle size, in mm, that has 60% passing)
- Specific Surface Area

11. Precision and Bias

11.1 Precision—No precision has been established for this test.

11.2 Bias—No bias can be established because no reference material is available for this test.

PROPOSED STANDARD METHOD OF TEST FOR RESISTANCE OF ABRASION OF COARSE AGGREGATE TO DEGRADATION BY ABRASION IN THE MICRO-DEVAL APPARATUS

1. Scope

1.1 This method covers a procedure for testing sizes of coarse aggregate from 9.5 mm to 19.0 mm for resistance to abrasion using the Micro-Deval apparatus.

2. Reference Documents

2.1 AASHTO Standards:
- T 27 Standard Method for Sieve Analysis of Fine and Coarse Aggregates
- M 92 Standard Specification for Wire-Cloth Sieves for Testing Purposes

2.2 Ontario Standards:
- LS-618 Method of Test for the Resistance of Coarse Aggregate to Degradation by Abrasion in the Micro-Deval Apparatus

3. Summary of Method

3.1 The Micro-Deval test is a measure of abrasion resistance and durability of mineral aggregates resulting from a combination of actions, including abrasion and grinding with steel balls in the presence of water. A sample with standard grading is initially soaked for not less than 1 hour. The sample is then placed in a jar mill with 2.0 l of water and an abrasive charge consisting of 5,000 g of 9.5 mm diameter steel balls. The jar, aggregate, water, and charge are revolved at 100 rpm for 2 hours. The sample is then washed and oven dried. The loss is the amount of material passing the 1.18 mm sieve expressed as a percent by mass of the original sample.

4. Significance and Use

4.1 The Micro-Deval test is a testing of coarse aggregates to determine their abrasion loss in the presence of water and an abrasive charge. It furnishes information helpful in judging the toughness/abrasion resistance and durability/soundness of coarse aggregate subject to abrasion and weathering action when adequate information is not available from service records.

5. Description of Terms

5.1 Constant Mass—Test samples dried at a temperature of 110 ± 5°C to a condition such that it will not lose more than 0.1 percent moisture after 2 hours of drying. Such a condition of dryness can be verified by weighing the sample before and after successive 2-hour drying periods. In lieu of such a determination, samples may be considered to have reached constant mass when they have been dried at a temperature of 110 ± 5°C for an equal or longer period than that previously found adequate for producing the desired constant mass condition under equal or heavier loading conditions of the oven.

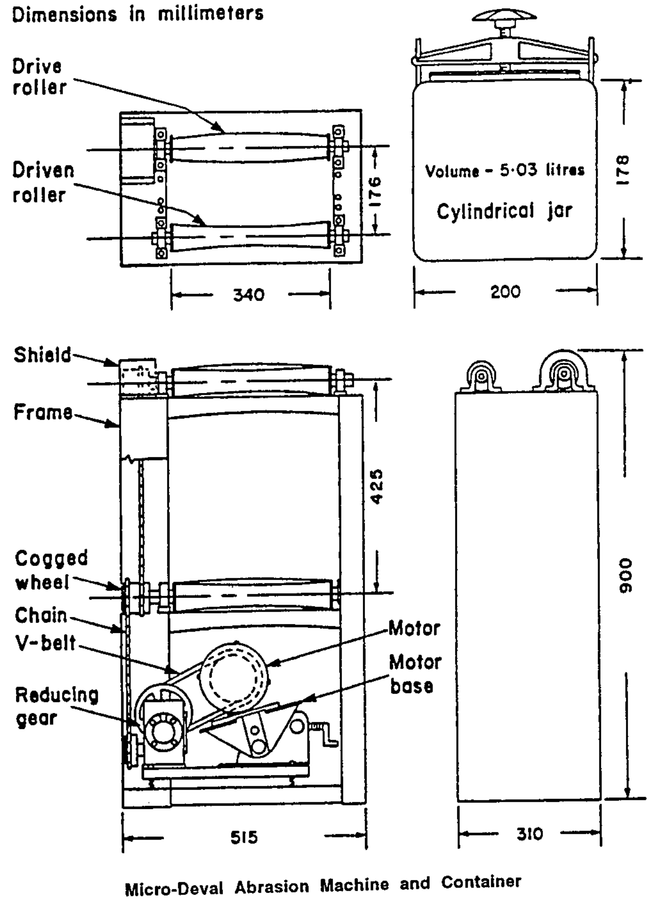

Figure D-3.

6. Apparatus

6.1 Micro-Deval Abrasion Machine—A jar rolling mill capable of running at 100 ± 5 rpm (Figure D-3).

6.2 Containers—Stainless steel, Micro-Deval abrasion jars having a 5-liter capacity with a rubber ring in the rotary locking cover. Internal diameter = 194 mm, internal height = 170 mm. The inside and outside surfaces of the jars shall be smooth and have no observable ridges or indentations.

6.3 Abrasion Charge—Stainless steel balls are required. These shall have a diameter of 9.5 ± 0.5 mm. Each jar requires a charge of 5,000 ± 5 g of balls.

6.4 Sieves—Sieves with square openings, and of the following sizes conforming to AASHTO M 92 specifications: 19.0 mm, 16.0 mm, 13.2 mm, 9.5 mm, 6.7 mm, 4.75 mm, 1.18 mm

6.5 Oven—The oven shall be capable of maintaining a temperature of 110 ± 5°C.

6.6 Balance—A balance or scale accurate to 1.0 g.

6.7 Laboratory Control Aggregate—A supply of standard 'Pittsburg' coarse aggregate available from the Soil and Aggregates Section, Ministry of Transportation, 1201 Wilson Avenue, Downsview, Ontario, M3M 1 J8, Canada.

7. Test Sample

7.1 The test sample shall be washed and oven-dried at 110 ± 5°C to substantially constant weight, separated into individual size fractions, and recombined to the grading as shown in section 7.2 below.

7.2 Aggregate for the test shall normally consist of material passing the 19.0 mm sieve, retained on the 9.5 mm sieve. An oven dry sample of 1,500 ± 5 g shall be prepared as follows:

Passing	*Retained*	*Mass*
9.0 mm	16.0 mm	375 g
16.0 mm	13.2 mm	375 g
13.2 mm	9.5 mm	750 g

7.3 In cases where the maximum nominal size of the coarse aggregate is less than 16.0 mm, a sample of 1,500 ± 5 g shall be prepared as follows:

Passing	*Retained*	*Mass*
13.2 mm	9.5 mm	750 g
9.5 mm	6.7 mm	375 g
6.7 mm	4.75 mm	750 g

8. Test Procedure

8.1 Prepare a representative 1,500 ± 5 sample. Record the Mass 'A' to the nearest 1.0 g.

8.2 Saturate the sample in 2.0 ± 0.05 l of tap water (temperature 20 ± 5°C) for a minimum of 1 hour.

8.3 Place the sample in the Micro-Deval abrasion container with 5,000 ± 5 g of steel balls and the water. Place the Micro-Deval container on the machine.

NOTE 1—It is permissible to saturate the sample in the water within the Micro-Deval abrasion container rather than in a separate container.

8.4 Run the machine at 100 ± 5 rpm for 2 hours ± 1 minute.

8.5 Carefully pour the sample over two superimposed sieves: 4.75 mm and 1.18 mm. Take care to remove all of the sample from the stainless steel jar. Wash the retained material with water (a hand-held spray will be found useful) until the washing water is clear. Remove the stainless steel balls using a magnet or other suitable means.

8.6 Combine the material retained on the 4.75 mm and 1.18 mm sieves, being careful not to lose any material.

8.7 Oven dry the sample to constant mass at 110 ± 5°C.

8.8 Weigh the sample to the nearest 1.0 g. Record the Mass 'B.'

9. Calculations

9.1 Calculate the Micro-Deval abrasion loss, as follows, to the nearest 0.1 percent

$$\text{Percent Loss} = (A - B) / A * 100$$

10. Use of Laboratory Control Aggregate

10.1 Every 10 samples, but at least every week in which a sample is tested, a sample of the standard reference aggregate shall also be tested. The material shall be taken from a stock supply and prepared according to section 6.7.

10.2 Control Chart Use: The percent loss of the last twenty samples of reference material shall be plotted on a control chart in order to monitor the variation in results (Figures D-4 and D-5).

11. Precision

11.1 As noted in Ontario Test Method LS-619, the multi-laboratory coefficient of variation has been found to be 10.0 percent and 6.4 percent for coarse aggregates with mean losses of 5 percent and 12 percent respectively. Therefore, the coefficient of variation conducted for two samples of the same material tested in two experienced laboratories should only differ from each other by 28 percent and 18 percent (D2S) one time in twenty for coarse aggregate with mean losses of 5 percent and 12 percent respectively.

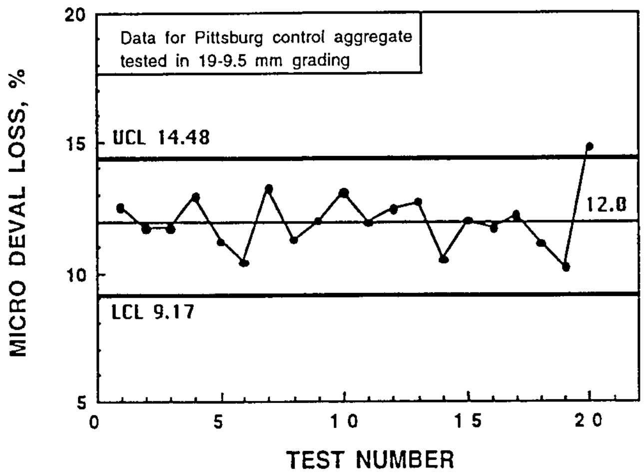

Figure D-4. Percent loss for 20 samples (19-9.5 mm grading).

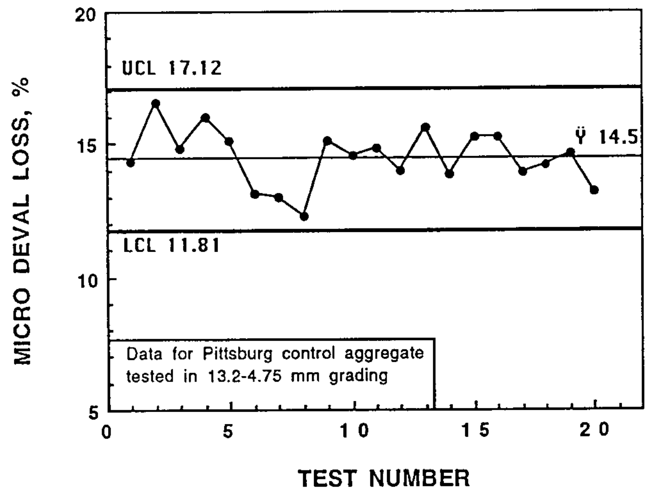

Figure D-5. Percent loss for 20 samples (13.2-4.75 mm grading).